



Investigation of the Performance of the New Orleans Flood Protection Systems in Hurricane Katrina on August 29, 2005

Volume I: Main Text and Executive Summary

by

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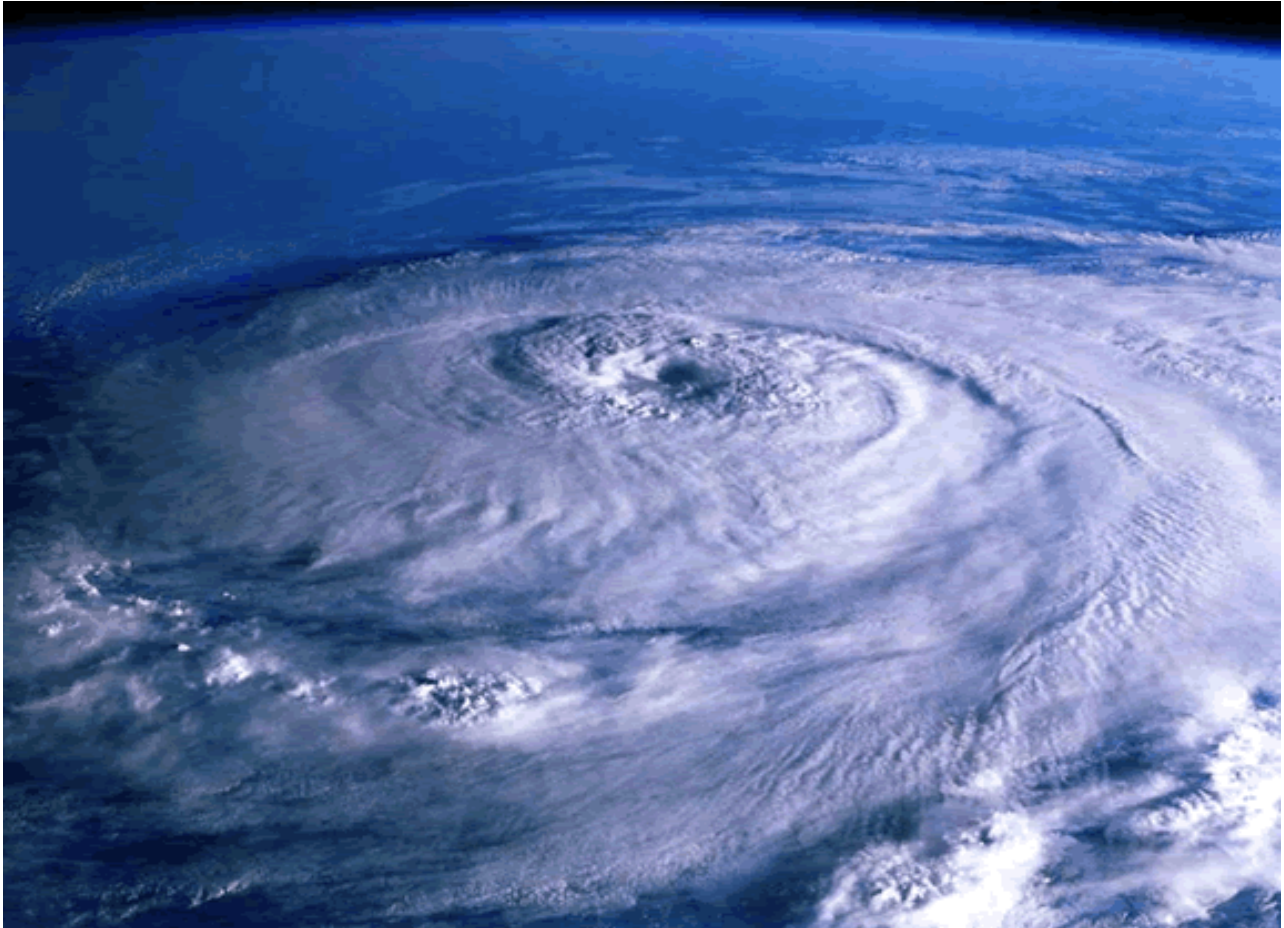
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This report contains the observations and findings of an investigation by an independent team of professional engineers and researchers with a wide array of expertise. The materials contained herein are the observations and professional opinions of these individuals, and do not necessarily reflect the opinions or endorsement of any other group or agency.

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This report is dedicated to the people of the greater New Orleans region;
to those that perished, to those that lost friends and loved ones,
and to those that lost their homes, their businesses, their place of work,
and their community.

New Orleans has now been flooded by hurricanes six times
over the past century; in 1915, 1940, 1947, 1965, 1969 and 2005.

It must be our goal that it not be allowed to happen again.

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Basin Foundation, New Orleans.	G-23
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EXECUTIVE SUMMARY

This report presents the results of an investigation of the performance of the New Orleans regional flood protection system during and after Hurricane Katrina, which struck the New Orleans region on August 29, 2005. This event resulted in the single most costly catastrophic failure of an engineered system in history. Current damage estimates at the time of this writing are on the order of \$100 to \$200 billion in the greater New Orleans area, and the official death count in New Orleans and southern Louisiana at the time of this writing stands at 1,293, with an additional 306 deaths in nearby southern Mississippi. An additional approximately 300 people are currently still listed as “missing”; it is expected that some of these missing were temporarily lost in the shuffle of the regional evacuation, but some of these are expected to have been carried out into the swamps and the Gulf of Mexico by the storm’s floodwaters, and some are expected to be recovered in the ongoing sifting through the debris of wrecked homes and businesses, so the current overall regional death count of 1,599 is expected to continue to rise a bit further. More than 450,000 people were initially displaced by this catastrophe, and at the time of this writing more than 200,000 residents of the greater New Orleans metropolitan area continue to be displaced from their homes by the floodwater damages from this storm event.

This investigation has targeted three main questions as follow: (1) What happened?, (2) Why?, and (3) What types of changes are necessary to prevent recurrence of a disaster of this scale again in the future?

To address these questions, this investigation has involved: (1) an initial field reconnaissance, forensic study and data gathering effort performed quickly after the arrival of Hurricanes Katrina (August 29, 2005) and Rita (September 24, 2005), (2) a review of the history of the regional flood protection system and its development, (3) a review of the challenging regional geology, (4) detailed studies of the events during Hurricanes Katrina and Rita, as well as the causes and mechanisms of the principal failures, (4) studies of the organizational and institutional issues affecting the performance of the flood protection system, (5) observations regarding the emergency repair and ongoing interim levee reconstruction efforts, and (6) development of findings and preliminary recommendations regarding changes that appear warranted in order to prevent recurrence of this type of catastrophe in the future.

In the end, it is concluded that many things went wrong with the New Orleans flood protection system during Hurricane Katrina, and that the resulting catastrophe had its roots in three main causes: (1) a major natural disaster (the Hurricane itself), (2) the poor performance of the flood protection system, due to localized engineering failures, questionable judgments, errors, etc. involved in the detailed design, construction, operation and maintenance of the system, and (3) more global “organizational” and institutional problems associated with the governmental and local organizations responsible for the design, construction, operation, maintenance and funding of the overall flood protection system.

After eight months of detailed study, a much clearer picture has now emerged regarding the causes and mechanisms of this catastrophe. Many of the findings of this study represent a different view of key elements of this event than has been publicly presented to date.

Hurricane Katrina was a large hurricane, and its arrival at New Orleans represented the root cause of a natural disaster. This disaster grew to a full blown catastrophe, however, principally due to the massive and repeated failure of the regional flood protection system and the consequent flooding of approximately 85% of the greater metropolitan area of New Orleans.

As Hurricane Katrina initially approached the coast, the resulting storm surge and waves rose over the levees protecting much of a narrow strip of land on both sides of the lower Mississippi River extending from the southern edge of New Orleans to the Gulf of Mexico. Most of this narrow protected zone, Plaquemines Parish, was massively inundated by the waters of the Gulf.

The eye of the storm next proceeded to the north, on a path that would take it just slightly to the east of New Orleans.

Hurricane Katrina has been widely reported to have overwhelmed the eastern side of the New Orleans flood protection system with storm surge and wave loading that exceeded the levels used for design of the system in that area. That is a true statement, but it is also an incomplete view. The storm surge and wave loading at the eastern flank of the New Orleans flood protection system was not vastly greater than design levels, and the carnage that resulted owed much to the inadequacies of the system as it existed at the time of Katrina's arrival. Some overtopping of levees along the eastern flank of the system (along the northeastern frontage of the St. Bernard and Ninth Ward protected basin, and at the southeast corner of the New Orleans East protected basin), and also in central areas (along the GIWW channel and the IHNC channel) was inevitable given the design levels authorized by Congress and the surge levels produced in these areas by the actual storm. It does not follow, however, that this overtopping had to result in catastrophic failures and breaching of major portions of the levees protecting these areas, nor the ensuing catastrophic flooding of these populous areas.

The northeast flank of the St. Bernard/Ninth Ward basin's protecting "ring" of levees and floodwalls was incomplete at the time of Katrina's arrival. The critical 11 mile long levee section fronting "Lake" Borgne (which is actually a Bay, connected directly to the Gulf of Mexico) was being constructed in stages, and funding appropriation for the final stage had long been requested by the U.S. Army Corps of Engineers (USACE), but this did not arrive before Katrina struck; as a result large portions of this critical levee frontage were several feet below final design grade. In addition, an unfortunate decision had been made to use local dredge spoils from the excavation of the adjacent MRGO channel for construction of major portions of the

levees along this frontage. The result was that major portions of these levees were comprised of highly erodeable sand and lightweight shell sand fill.

When the storm surge arrived, massive portions of these levees eroded catastrophically and the storm surge passed through this frontage while still on the rise, crossed an open swamp area that should have safely absorbed most of the overtopping flow from the outer levees (if they had not catastrophically eroded), and it then crossed easily over a secondary levee of lesser height that had not been intended to face a storm surge largely undiminished by the minimal interference of the too rapidly eroded outer levees fronting Lake Borgne. The resulting carnage in St. Bernard Parish was devastating, as the storm surge rapidly filled the protected basin to an elevation of approximately +12 feet above sea level; deeply inundating even neighborhoods with ground elevations well above sea level in this area.

The storm surge swelled waters of Lake Borgne also passed over and then through a length of levees at the southeast corner of the New Orleans East protected basin. Here too, the levees fronting Lake Borgne had been constructed primarily using materials dredged from the excavation of an adjacent channel (the GIWW channel), and these levees also contained major volumes of highly erodeable sands and lightweight shell sands. These levees were also massively eroded, and produced the principal source of flooding that eventually inundated the New Orleans East protected area. Here again there was an area of undeveloped swampland behind the outer levees that might have absorbed the brunt of any overtopping flow, and a secondary levee of lesser height was in place behind this swampland that might then have prevented catastrophic flooding of the populous areas of New Orleans East. This secondary levee was not able to resist the massive flows resulting from the catastrophic erosion of the highly erodeable section of the Lake Borgne frontage levee, however, and the floodwaters passed over the secondary levee and began the filling of the New Orleans East protected basin.

The catastrophic erosion of these two critical levee frontages need not have occurred. These frontages could instead have been constructed using well compacted clay fill with good resistance to erosion, and they could have been further armored in anticipation of the storm surge and wave loading from Lake Borgne. The levee at the northeast edge of St. Bernard Parish could have been completed in a more timely manner. The result would have been some overtopping, but not catastrophic erosion and uncontrolled breaching of these critical frontages. Some flooding and damage would have been expected, but it need not have been catastrophic.

The storm surge swollen waters of Lake Borgne next passed laterally along the east-west trending GIWW/MRGO channel to its intersection at a "T" with the north-south oriented IHNC channel, overtopping levees along both banks to a limited degree. This produced an additional breach of a composite earthen levee and concrete floodwall section along the southern edge of New Orleans East, adding additional uncontrolled inflow to this protected basin. This failure could have been prevented at little incremental cost if erosion protection (e.g. a concrete splash pad, or similar) had been emplaced along the back side of the concrete floodwall at the levee crest, but the USACE

felt that this was precluded by Federal rules and regulations regarding authorized levels of protection.

The surge next raised the water levels within the IHNC channel, and produced a number of failures on both the east and west banks. Two major failures occurred on the east side of the IHNC, at the west edge of the Ninth Ward. Overtopping occurred at both of these locations, but this was not the principal cause of either of these failures. Both failures were principally due to underseepage flows that passed beneath the sheetpile curtains supporting the concrete floodwalls at the crests of the levees. Like many sections of the flood protection system, these sheetpiles were too shallow to adequately cut off, and thus reduce, these underseepage flows. The result was two massive breaches that devastated the adjacent Ninth Ward neighborhood, and then pushed east to meet with the floodwaters already rapidly approaching from the east from St. Bernard Parish as a result of the earlier catastrophic erosion of the Lake Borgne frontage levees.

Several additional breaches also occurred farther north on the east side of the IHNC fronting the west side of New Orleans East, but these were relatively small features and they just added further to the uncontrolled flows that were now progressively filling this protected basin. These breaches occurred mainly at junctures between adjoining, dissimilar levee and floodwall sections, and represented good examples of widespread failure to adequately engineer these “transitions” between sections of the regional flood protection system.

Several breaches occurred on the west side of the IHNC, and these represented the first failures to admit uncontrolled floodwaters into the main metropolitan (downtown) protected area of New Orleans. These features did not scour and erode a path below sea level, however, so they admitted floodwaters for a number of hours and then these inflows ceased as the storm surge in the IHNC eventually subsided. Only 10% to 20% of the floodwaters that eventually inundated a majority of the main (downtown) New Orleans protected basin entered through these features.

These failures and breaches on the west side of the IHNC all appear to have been preventable. One failure was the result of overtopping of an I-wall, with the overtopping flow then eroding a trench in the earthen levee crest at the inboard side of the floodwall. This removal of lateral support unbraced the floodwall, and it was pushed over laterally by the water pressures from the storm surge on the outboard side. Here again the installation of erosional protection (e.g. concrete splash pads or similar) might have prevented the failure.

The other failures in this area occurred at “transitions” between disparate levee and floodwall sections, and/or at sections where unsuitable and highly erodible lightweight shell sand fills had been used to construct levee embankments. Here, again, these failures were as much the result of design choices and/or engineering and oversight issues as the storm surge itself.

As the eye of the hurricane next passed to the northeast of New Orleans, the counterclockwise swirl of the storm winds produced a storm surge against the southern edge of Lake Pontchartrain. This produced additional temporary overtopping of a long section of levee and floodwall at the west end of the lakefront levees of New Orleans east, behind the old airport, adding further to the flows that were progressively filling this protected basin.

The surge against the southern edge of Lake Pontchartrain also elevated the water levels within three drainage canals at the northern edge of the main metropolitan (downtown) New Orleans protected basin, and this would produce the final, and most damaging, failures and flooding of the overall event.

The three drainage canals should not have been accessible to the storm surge. The USACE had tried for many years to obtain authorization to install floodgates at the north ends of the three drainage canals that could be closed to prevent storm surges from raising the water levels within the canals. That would have been the superior technical solution. Dysfunctional interaction between the local Levee Board (who were responsible for levees and floodwalls, etc.) and the local Water and Sewerage Board (who were responsible for pumping water from the city via the drainage canals) prevented the installation of these gates, however, and as a result many miles of the sides of these three canals had instead to be lined with levees and floodwalls.

The lining of these canals with levees topped with concrete floodwalls was rendered very challenging due to (a) the difficult local geology of the foundation soils, and (b) the narrow right of way (or available “footprint”) for these levees. As a result of the decision not to install the floodgates, the three canals represented potentially vulnerable “daggers” pointed at the heart of the main metropolitan New Orleans protected basin. Three major breaches would occur on these canals; two on the London Avenue Canal and one on the 17th Street Canal. All three of these breaches eroded and scoured rapidly to well below sea level, and these three major breaches were the source of approximately 80% of the floodwaters that then flowed into the main (downtown) protected basin over the next three days, finally equilibrating with the still slightly elevated waters of Lake Pontchartrain on Thursday, September 1.

The central canal of the three, the Orleans Canal, did not suffer breaching, but a section of floodwall topping the earthen levee approximately 300 feet in length near the south end of the canal had been left incomplete, again as a result of dysfunctional interaction between the local levee board and the water and sewerage board. This effectively reduced the level of protection for this canal from about +12 to +13 feet above sea level (the height of the tops of the floodwalls lining the many miles of the canal) to an elevation of about +6 to +7 feet above sea level (the height of the earthen levee crest along the 300 foot length where the floodwall that should have topped this levee was omitted). As a result of the missing floodwall section, flow passed through this “hole” and began filling the heart of the main New Orleans protected basin. This flow eventually ceased as the storm surge subsided, and so was locally damaging but not catastrophic.

The three breaches on the 17th Street and London Avenue canals were catastrophic. None of these failures were the result of overtopping; surge levels in all three drainage canals were well below the design levels, and well below the tops of the floodwalls. Two of these breaches were the result of stability failures of the foundation soils underlying the earthen levees and their floodwalls, and the third was the result of underseepage passing beneath the sheetpile curtain and resultant catastrophic erosion near the inboard toe of the levee that eventually undermined the levee and floodwall.

A large number of engineering errors and poor judgements contributed to these three catastrophic design failures, as detailed in Chapter 8. In addition, a number of these same problems appear to be somewhat pervasive, and call into question the integrity and reliability of other sections of the flood protection system that did not fail during this event. Indeed, additional levee and floodwall sections appear to have been potentially heading towards failure when they were “saved” by the occurrence of the three large breaches (which rapidly drew down the canal water levels and thus reduced the loading on nearby levee and floodwall sections.)

The New Orleans regional flood protection system failed at many locations during Hurricane Katrina, and by many different modes and mechanisms. This unacceptable performance was to a large degree the result of more global underlying “organizational” and institutional problems associated with the governmental and local organizations jointly responsible for the design, construction, operation, and maintenance of the flood protection system, including provision of timely funding and other critical resources.

Our findings to date indicate that no one group or organization had a monopoly on responsibility for the catastrophic failure of this regional flood protection system. Many groups, organizations and even individuals had a hand in the numerous failures and shortcomings that proved so catastrophic on August 29th. It is a complex situation, without simple answers.

It is not without answers and potential solutions, however, just not simple ones. There is a need to change the process by which these types of large and critical protective systems are created and maintained. It will not be feasible to provide an assured level of protection for this large metropolitan region without first making significant changes in the organizational structure and interactions of the national and more local governmental bodies and agencies jointly responsible for this effort. Significant changes are also needed in the engineering approaches and procedures used for many aspects of this work, and there is a need for interactive and independent expert technical oversight and review as well. In numerous cases, it appears that such review would have likely caught and challenged errors and poor judgements (both in engineering, and in policy and funding) that led to failures during Hurricane Katrina.

Simply updating engineering procedures and design manuals will not provide the needed level of assurance of safety of the population and properties of this major metropolitan region. Design procedures and standards employed for many elements of

the flood protection system can be traced back to initial development and use for design and construction of levees intended for protection of largely unpopulated agrarian land, not a major urban region. Design levels of safety and reliability were nowhere near those generally used for major dams; largely because dams are considered to pose a potential risk to large populations. There are few U.S. dams that pose risk to populations as large as the greater New Orleans region, however, and it is one of the recommendations of this study that standards and policies much like those used for “dams” should be adopted for levee systems protecting such regions.

Simply addressing engineering design standards and procedures is unlikely to be sufficient to provide a suitably reliable level of protection. There is also a need to resolve dysfunctional relationships between federal and more local government, and the federal and local agencies responsible for the actual design, construction and maintenance of such flood protection systems. Some of these groups need to enhance their technical capabilities; a long-term expense that would clearly represent a prudent investment at both the national and local level, given the stakes as demonstrated by the losses in this recent event. Steady commitment and reliable funding, shorter design and construction timeframes, clear lines of authority and responsibility, and improved overall coordination of disparate system elements and functions are all needed as well.

And there is some urgency to all of this. The greater New Orleans regional flood protection system was significantly upgraded in response to flooding produced by Hurricane Betsy in 1965. The improved flood protection system was intended to be completed in 2017, fully 52 years after Betsy’s calamitous passage. The system was incomplete when Katrina arrived. As a nation, we must manage to dedicate the resources necessary to complete projects with such clear and obvious ramifications for public safety in a more timely manner.

New Orleans has now been flooded by hurricanes six times over the past century; in 1915, 1940, 1947, 1965, 1969 and 2005. It should not be allowed to happen again.

THE INVESTIGATION TEAM

The University of California at Berkeley led Independent Levee Investigation Team (ILIT) grew through the course of this investigation, and eventually numbered 35 very dedicated and accomplished individuals.

The team included a large number of leading experts across a diverse range of fields. Team members came from six states, and they came from universities, private engineering firms, and state and federal agencies.

As a group, the investigation team had very impressive prior experience with forensic studies of major disasters and catastrophes. For example, the team members had previously investigated 12 major earthquakes and 8 major hurricanes (both domestic and foreign), 14 dam failures, more than a dozen levee failures, numerous landslides, one tsunami, the pivotal Kettleman Hills waste landfill failure, the Challenger and Columbia space shuttle disasters, the Exxon Valdez tanker disaster, and a number of major offshore pipeline and oil platform failures. They are well experienced with the carnage and disarray of disasters, and with the unforgettable smell of death. They are also well experienced at the delicate and deliberate art and science of piecing their way through the devastation, carefully and professionally, and figuring out what had happened, and why; the art and science of engineering forensics.

The calibre of these assembled experts is such that we could never possibly have afforded to hire them. Instead, excepting a handful of graduate research students who worked for very low wages, these world class experts all volunteered, and they worked pro bono (for free.) They did this for the intellectual challenge, for the camaraderie of a very special group of accomplished colleagues, for the chance to make a positive difference, because it was important, and most importantly because it was the right and necessary thing to do.

The pages that follow list the names and affiliations of the members of the Independent Levee Investigation Team. I have had the opportunity to work on a number of investigations of major catastrophes and disasters, but I have never worked with a finer group. They are all heroes in my book.

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The authors also wish to express their gratitude to the U.S. Army Corps of Engineers (USACE) for their considerable assistance with numerous elements of this work. Their field investigation team from the Engineer Research Development Center (ERDC) in Vicksburg hosted and assisted our own field investigation team in the critical early days of late September and early October. The USACE has also posted massive amounts of background documents on their website, and this has been an invaluable resource. The USACE, and the Interagency Performance Evaluation Team (IPET) have graciously shared much of their field and laboratory data, and we have done the same. This positive sharing and collaboration helps everyone by providing the best possible basis for study and analysis of this event.

We are also deeply grateful to the honorable men and women of the USACE who have taken extra measures to help to provide additional documents, data and insight. Many of these prefer not to be named, but their dedication to service of the greater public good in this difficult situation has been admirable.

We are deeply grateful to the members of the State of Louisiana's independent investigation team, Team Louisiana, for their tremendous efforts and dogged persistence under very difficult circumstances, and for their generous mutual sharing of data and insights throughout this investigation. This team consists of Dr. Ivor Van Heerden, Dr. Paul Kemp and Dr. Hassan Mashriqui (all from the Louisiana State University Hurricane Research Center), Billy Prochaska and Dr. Lou Cappozzoli (both local geotechnical consultants), and Art Theis (retired head of the Louisiana Department of Public Works.) The people of Louisiana, and the nation, owe these gentlemen a great debt as their persistent efforts have, time and again, produced critical data and insights that would not otherwise have been available.

We are also grateful to the members of the field investigation team of the American Society of Civil Engineers, who jointly formed a combined team with ours in the urgent initial post-event field studies when it was of vital importance to gather all possible data and observations while (fully necessary) emergency repair operations were already damaging and burying critical evidence. This was a very strong field forensics team, and their collaboration both in the field and in the subsequent preparation of an initial Preliminary Report which was issued in early November of 2005, was of great value.

Finally we are deeply grateful to the many others who will remain anonymous, but who have assisted by providing information, data, background history and other information that might otherwise not have been available.

A great many people gave generously of themselves, their time, and their expertise to assist these studies. It was important, and we are profoundly grateful.

CHAPTER ONE: INTRODUCTION AND OVERVIEW

1.1 Introduction

This report presents the results of an investigation of the performance of the New Orleans regional flood protection system during and after Hurricane Katrina, which struck the New Orleans region on August 29, 2005. This event resulted in the single most costly catastrophic failure of an engineered system in history. Current damage estimates at the time of this writing are on the order of \$100 to \$200 billion in the greater New Orleans area, and the official death count in New Orleans and southern Louisiana at the time of this writing stands at 1,293 with an additional 306 deaths in nearby southern Mississippi. An additional approximately 300 people are currently still listed as “missing”; it is expected that some of these missing were temporarily lost in the shuffle of the regional evacuation, but some of these are expected to have been carried out into the swamps and the Gulf of Mexico by the storm’s floodwaters, and some are expected to be recovered in the still ongoing sifting through the debris of wrecked homes and businesses, so the current overall regional death count of 1,599 is expected to continue to rise a bit further. More than 450,000 people were initially displaced by this catastrophe, and at the time of this writing, more than 200,000 residents of the greater New Orleans metropolitan area continue to be displaced from their homes by the floodwater damages from this storm event.

This investigation targets three main questions as follow: (1) What happened? (2) Why? and (3) What types of changes are necessary to prevent recurrence of a disaster of this scale again in the future?

To address these questions, this investigation has involved: (1) an initial field reconnaissance, forensic study and data gathering effort performed quickly after the arrival of Hurricanes Katrina (August 29, 2005) and Rita (September 24, 2005), (2) a review of the history of the regional flood protection system and its development, (3) a review of the challenging regional geology, (4) detailed studies of the events during Hurricanes Katrina and Rita, as well as the causes and mechanisms of the principal failures, and studies of sections that performed successfully as well, (5) studies of the organizational and institutional issues affecting the performance of the flood protection system, (6) observations regarding the emergency repair and ongoing interim levee reconstruction efforts, and (7) development of findings and preliminary recommendations regarding changes that appear warranted in order to prevent recurrence of this type of catastrophe in the future.

1.2 Initial Post-Event Field Investigations

A critical early stage of this investigation was the initial field investigations performed by collaborating teams of engineers and scientists in the wake of the passage of Hurricane Katrina, to study performance of the regional flood protection system and the resulting flooding that occurred in the New Orleans area. The principal focus of these efforts was to capture perishable data and observations related to the performance of flood protection system before they were lost to ongoing emergency response and repair operations.

Several independent investigation teams jointly pooled their efforts in order to capture as much data as possible in the precious timeframe available. The two principal participating teams were from the University of California at Berkeley (UC Berkeley) which included a number of colleagues from other firms and institutions, and a team from the American Society of Civil Engineers (ASCE) organized by its Geo-Institute and by its Coasts, Oceans, Ports, and Rivers Institute. A team from Louisiana State University's Hurricane Research Center (LSU/HRC) also accompanied the field investigation teams during their first week of investigations. These teams were accompanied and assisted in the field by members of the U.S. Army Corps of Engineers (USACE) levee investigation team from the Engineer Research and Development Center (ERDC). All of these investigative teams shared data and findings freely and openly, and the mutual pooling of talents and expertise greatly benefited all as it enabled the field teams to gather more data in the critical days available.

These initial field investigations occurred over a span of approximately three weeks, from September 26 through October 15, 2005, and the preliminary observations and findings were presented in a report jointly authored by the UC Berkeley-led field investigation team and the ASCE field investigation team (Seed, et al.; November 15, 2005.)

1.3 Current Studies and Investigations

Subsequent to these initial field investigations, three main investigations have been carried forward. The largest of these is the U.S. Army Corps of Engineers' own internal investigation, the Interagency Performance Evaluation Team (IPET) study. The IPET study is by far the largest of the three investigations, and has a budget of approximately \$20 million. The American Society of Civil Engineers (ASCE) has been hired, for an additional \$2 million, to form a review panel (called the External Review Panel, E.R.P.) to review the results of the IPET studies. This ASCE review panel works and consults closely with the IPET studies and is focused specifically on reviewing the IPET investigation efforts, data and findings. The National Research Council (NRC) has also been hired, by the Department of Defense, to provide an additional review of the IPET studies after the ASCE's E.R.P. completes its task. This NRC review panel has announced its intention of reviewing input from all investigation teams and efforts as part of this task.

The IPET study is narrowly focused and constrained in its first year to consideration and study of only "what happened" in a strictly physical sense; it is specifically not to address underlying faults or to assign "responsibility" in its initial studies (Final Draft Report due to the ASCE review panel on May 15, 2006, and Final Report due on June 1, 2006), but rather to wait and study "organizational issues", "human factors", etc. during the following year.

The second investigation team moving forward is Team Louisiana, representing the interests of the State of Louisiana in performing an investigation independent of the USACE. Team Louisiana is led by Dr. Ivor Van Heerden, and its core is formed by a number of his colleagues from the Louisiana State University (LSU) Hurricane Research Center (LSU/HRC), with additional members from a number of local engineering consulting firms and state organizations. Team Louisiana does not have the massive funding or manpower of the IPET team, but they are strongly motivated and have worked very hard and well given

their logistical limitations and the difficult situation of the region (which has directly affected some of the team's members, as well as many of their friends and colleagues.)

The third investigation team moving forward is our own UC Berkeley-led Independent Levee Investigation Team (ILIT). Our budget is also not as large as that of the IPET study, and currently stands at approximately \$350,000. We have, however, managed to assemble a team of 37 outstanding engineers and researchers. Pages “xxv” through “xxvi” describe the team. As a group, the conjugate forensic experience in prior investigations of numerous major engineering and natural disasters is very impressive. This is an amazingly strong team, and we could never possibly have afforded to hire them within our small budget. These leading experts have, instead, volunteered to work for free (pro bono), and our budget is thus devoted instead towards covering travel expenses, field borings and sampling, and laboratory testing, etc. We have elected to decline proffered offers of additional funding, as it appears important that our investigation team maintain its demonstrable independence and neutrality in these studies.

1.4 Organization of this Report

This report presents the results of studies directed towards answering three main sets of questions as follow:

1. **What happened?** What events transpired during Hurricane Katrina and during its aftermath? How did the regional flood protection system perform? What were the successes, and what were the shortcomings and failures? What mechanisms and forces, etc., led to these performances?
2. **Why did this happen?** What were the underlying issues that led to the observed performance of the system elements? What were the influences of regional and local geology? How did the history of the evolution of the flood protection system contribute to its performance? What were the design assumptions, engineering studies and analyses, etc., and what effect did these have on the performance of the system elements? What over-arching organizational, institutional, political and funding issues may have played a role?
3. **What can be done to ensure that a similar catastrophe does not recur in the future?** This report presents preliminary findings and recommendations regarding changes in organization of the overall governmental/institutional “system” responsible for the conception, design, construction, operation and maintenance of the complex regional flood defense system, as well as the making of political decisions regarding levels of protection to be provided, and the provision of funding to support the creation and operation/maintenance of such a system. This report also presents preliminary findings and recommendations regarding a number of focused areas for improvement of the conceptual design, analysis and engineering design, and construction and maintenance of such a system.

In the end, it is concluded that many things went wrong with the New Orleans flood protection system during Hurricane Katrina, and that the resulting catastrophe had its roots in

three main causes: (1) a major natural disaster (the Hurricane itself), (2) the poor performance of the flood protection system, due to localized engineering failures, questionable judgements, errors, etc. involved in the detailed design, construction, operation and maintenance of the system, and (3) more global “organizational” and institutional problems associated with the governmental and local organizations responsible for the design, construction, operation, maintenance and funding of the overall flood protection system.

Chapter 2 presents an overview of the principal events that occurred during and after the arrival of Hurricane Katrina in the New Orleans area, with emphasis on the storm surge and wave loadings, and the resulting performance of the regional flood protection system.

Chapter 3 presents a summary overview of the challenging regional and local geology that so strongly affects the difficulties associated with the creation of regional flood protection systems, and their performance as well.

Chapter 4 presents a review of the history of the development of the New Orleans regional flood protection system. It is a truism of levees and flood protection that the fabric and history of a given region is usually closely interwoven within the fabric of the levees and flood protection systems that are created in that region.

Chapters 5 through 8 present the results of studies and analyses of the performance of the four main levee-protected areas principally affected by Hurricane Katrina. These chapters present overviews of the performance of the flood protection system in each of the four areas, of the flooding that occurred within each of these areas, and detailed analyses of the performance of critical sub-elements of the system within each area. These analyses include an investigation of the causes of critical failures, and the apparent reasons for these including both engineering/construction types of issues as well as organizational/institutional issues. These chapters also present observations, recommendations and findings related to some of the emergency post-hurricane repair and reconstruction efforts.

Chapter 9 presents the results of studies of issues associated with overtopping erosion and scour; a key phenomenon involved in both the successful and unsuccessful performances of numerous critical levee and floodwall sections throughout the region.

Chapter 10 briefly addresses a series of “other issues”, including a brief overview of design standards, observations regarding a number of recurrent issues that appear to be problematic throughout the regional flood protection system, performance assessment with regard to erosion and erodeability of placed fills, a brief overview of the performance of the pumping systems that “unwater” the protected areas of these studies, and observations and comments regarding the initial emergency levee and floodwall breach repair efforts, and the ongoing interim repair and reconstruction efforts, at a number of locations.

Chapter 11 presents a summary review of the engineering issues addressed in Chapters 2 through 10, and recommendations for changes in engineering and design practices to address these.

Chapters 12 through 14 examine a number of organizational and institutional issues that affected the performance of the regional flood protection systems during Hurricane Katrina. They also address recommendations for moving forward; recommendations for a number of changes to ensure that we never again have to study a catastrophe of this type and scale in southern Louisiana.

Chapter 12 begins with a review of background and history pertaining to these types of issues. Chapter 13 then presents a review and examination of critical organizational, institutional, political and funding issues that directly affected the performance of the New Orleans regional flood protection system, and also some of the post-hurricane repair and reconstruction efforts. These organizational/institutional issues had a dominant impact on the overall performance of the regional flood protection systems, and many of the problems that led to the catastrophic flooding of much of the greater New Orleans region can be traced directly (at least in large part) to these types of underlying issues.

Chapter 14 presents preliminary recommendations for changes that can and should be made in moving forward, in order to ensure that a catastrophe of this scale is never repeated in the future. The New Orleans regional flood protection system did not perform well in Hurricane Katrina. We can do better. This chapter presents recommendations for changes in specific engineering analysis and design procedures, conceptual design features and approaches, specific system elements, etc. This chapter also presents recommendations regarding changes in the overall system of governmental bodies, governmental agencies, outsourced (private sector) engineering and construction, local oversight agencies, and the regulations and procedures involved in the overall conception, design, construction, operation and maintenance of complex and regionally massive systems protecting vital public safety for populous regions such as this.

Finally, Chapter 15 presents a summary overview of these studies, and of the principal findings and recommendations.

1.5 Elevation Datum

There are a number of datums that have been and continue to be used for elevation references throughout the New Orleans Region. A good discussion of these is presented in the IPET Interim Report No. 2 (IPET; April 1, 2006). The situation is further confused as some regional benchmarks, which were considered stable, have recently been found to have instead subsided, so that elevations based on these require correction. In this present report, all elevations are stated in terms of local Mean Sea Level (MSL), which corresponds approximately to the NAVD88 (2004.65) datum. [This NAVD88 (2004.65) datum is currently thought to be within approximately 3-inches of Mean Sea Level in the New Orleans area.] All elevations in this report have been resolved, as best we were able with the information available, to this MSL (or approximately NAVD88; 2004.65) datum.

1.6 References

Seed, R. B., et al., "Preliminary Report on the Performance of the New Orleans Levee Systems in Hurricane Katrina on August 29, 2005", Report No. UCB/CITRIS – 05/01, November 17, 2005.

CHAPTER TWO: OVERVIEW OF HURRICANE KATRINA AND ITS AFTERMATH

2.1 Hurricane Katrina

The path of Hurricane Katrina's eye is shown in Figures 2.1 and 2.2. Hurricane Katrina crossed the Florida peninsula on August 25, 2005 as a Category 1 hurricane. It then entered the Gulf of Mexico, where it gathered energy from the warm Gulf waters, producing a hurricane that eventually reached Category 5 status on Sunday, August 28, shortly before making its second mainland landfall just to the east of New Orleans early on Monday, August 29, as shown in Figures 2.1 and 2.2. The Hurricane had weakened to a Category 4 level prior to landfall on the morning of August 29, and it weakened further as it came ashore.

Because the eye of this hurricane passed just slightly to the east of New Orleans, the hurricane imposed unusually severe wind loads and storm surges (and waves) on the New Orleans region and its flood protection systems.

2.2 Overview of the New Orleans Flood Protection Systems

Figure 2.3 shows the main study region. The City of New Orleans is largely situated between the Mississippi River, which passes along the southern edge of the main portion of the city, and Lake Pontchartrain, which fronts the city to the north. Lake Borgne lies to the east, separated from developed areas by open swampland. "Lake" Borgne is not really a lake at all; instead it is a bay as it is directly connected to the waters of the Gulf of Mexico. To the southeast of the city, the Mississippi River bends to the south and flows out through its delta into the Gulf of Mexico.

The flood protection system that protects the New Orleans region is organized as a series of protected basins or "protected areas", each protected by its own perimeter levee system, and these are "unwatered" by pumps.

As shown in Figures 2.4 and 2.5, there are four main protected areas that comprise the New Orleans flood protection system of interest. A number of additional levee-protected units also exist in this area, but the focus of these current studies is the four main protected areas shown in Figures 2.4 and 2.5. These were largely constructed under the supervision of the U.S. Army Corps of Engineers, to provide improved flood protection in the wake of the devastating flooding caused by Hurricane Betsy in 1965.

Figures 2.4 and 2.5 show the locations of most of the levee breaches and severely distressed (but non-breached, or only partially breached) levee sections covered by these studies. Levee breaches are shown with solid blue stars, and distressed sections as well as minor or partial breaches are indicated by red stars. The original base maps, and many of the stars, were graciously provided by the USACE (2005), and a number of additional blue and red stars have been added to the map in Figure 2.4 as a result of the studies reported herein. The yellow stars shown in these figures correspond to deliberate breaches made after Hurricane Katrina, to facilitate draining the flooded areas after the storm.

The pink shading in Figures 2.4 and 2.5 shows developed areas that were flooded, and the areas shaded with blue cross-hatching indicate undeveloped swamp land that was flooded. The deeper blue shading (near the east end of New Orleans East) denotes areas that still remained to be unwatered as late as September 28, 2005. As shown in these figures, approximately 85% of the metropolitan area of New Orleans was flooded during this event.

As shown in Figure 2.4, the Orleans East Bank (Metro Orleans) section is one contiguously protected section. This protected unit contains the downtown district, the French Quarter, the Garden District, and the “Canal” District. The northern edge of this protected area is fronted by Lake Pontchartrain on the north, and the Mississippi River passes along its southern edge. The Inner Harbor Navigation Canal (also locally known as “the Industrial Canal”) passes along the east flank of this protected section, separating the Orleans East Bank protected section from New Orleans East (to the northeast) and from the Lower Ninth Ward and St. Bernard Parish (directly to the east.) Three large drainage canals extend into the Orleans East Bank protected section from Lake Pontchartrain to the north, for the purpose of conveying water pumped north into the lake by large pump stations within the city. These canals, from west to east, are the 17th Street Canal, the Orleans Canal, and the London Avenue Canal.

A second protected section surrounds and protects New Orleans East, as shown in Figure 2.4. This protected section fronts Lake Pontchartrain along its north edge, and the Inner Harbor Navigation Canal (IHNC) along its west flank. The southern edge is fronted by the Mississippi River Gulf Outlet channel (MRGO) which co-exists with the Gulf Intracoastal Waterway (GIWW) along this stretch. The eastern portion of this protected section is currently largely undeveloped swampland, contained within the protective levee ring. The east flank of this protected section is fronted by additional swampland, and Lake Borgne is located slightly to the southeast.

The third main protected section contains both the Lower Ninth Ward and St. Bernard Parish, as shown in Figure 2.4. This protected section is also fronted by the Inner Harbor Navigation Canal on its west flank, and has the MRGO/GIWW channel along its northern edge. At the northeastern corner, the MRGO bends to the south (away from the GIWW channel) and fronts the boundary of this protected area along the northeastern edge. Open swampland occurs to the south and southeast. Lake Borgne occurs to the east, separated from this protected section by the MRGO channel and by a narrow strip of undeveloped marshland. The main urban areas occur within the southern and western portions of this protected area. The fairly densely populated Lower Ninth Ward is located at the west end, and St. Bernard Parish along approximately the southern half of the rest of this protected area. The northeastern portion of this protected section is undeveloped marshy wetland, as indicated in Figure 2.4. A secondary levee, operated and maintained by local levee boards, separates the undeveloped marshlands of the northeastern portions of this protected area from the Ninth Ward and St. Bernard Parish urban areas.

The fourth main protected area is a narrow, protected strip along the lower reaches of the Mississippi River heading south from St. Bernard Parish to the mouth of the river at the Gulf of Mexico, as shown in Figure 2.5. This protected strip, with “river” levees fronting the Mississippi River and a second, parallel set of “storm” levees facing away from the river forming a protected corridor less than a mile wide, serves to protect a number of small communities as well as utilities and pipelines. This protected corridor also provides protected access for workers, supplies and

gas and oil pipelines servicing the large offshore oil fields out in the Gulf of Mexico. This will be referred to in this report as “the Plaquemines Parish” levee protected zone.

The current perimeter levee and floodwall defense systems for these four protected areas were largely designed and constructed under the supervision of the U.S. Army Corps of Engineers in the wake of the catastrophic flooding caused by Hurricane Betsy of 1965. These flood protection improvements typically involved either new levee construction, or raising existing levee defenses and/or adding new floodwalls, to provide storm flood protection for higher elevations of storm surge waters (and waves) at locations throughout the region.

2.3 Overview of Flood Protection System Performance During Hurricane Katrina

2.3.1 Storm Surge During Hurricane Katrina

The regional flood protection system had been designed to safely withstand the storm surges and waves associated with the Standard Project Hurricane, which was intended to represent a scenario roughly “typical” of a rapidly moving Category 3 hurricane passing close to the New Orleans metropolitan region. Chapter 12 (Section 12.5.1) presents a more detailed discussion of the “Standard Project Hurricane”, and the criteria for which the regional flood protection system was designed. In simple terms, the system was intended to have been designed to safely withstand storm surge levels (plus waves) to specified elevations at various locations, as shown in Figures 2.6 and 2.7.

In general, the “Standard Project Hurricane” provided for design to safely withstand storm surge rises (plus waves) to prescribed elevations at various locations throughout the system. The levels selected correspond generally to the storm surge level (mean peak storm surge water elevation, without waves) associated with the “Standard Project Hurricane” conditions plus an additional allowance for most (but not always all) of expected additional wave run-up.

As shown in Figures 2.6 and 2.7, this resulted in a targeted protection level of about elevation +17 feet to +19 feet (MSL), or 17 to 19 feet above Mean Sea Level, at the eastern flank of the system, and + 13.5 feet to +18 feet (MSL) along much of the southern edge of Lake Pontchartrain. The storm surge levels within the various drainage canals and navigational channels varied, and the storm surge levels for design were typically on the order of Elev. + 14 feet to + 16 feet (MSL) along the GIWW and IHNC channels, and Elev. + 12.5 feet to + 14.5 feet (MSL) along the 17th Street, Orleans, and London Avenue Canals in the “Canal District”. There is some minor confusion as to the most recent “Standard Project Hurricane”, and the most recent storm surge design levels at some locations; the values indicated in Figure 2.6 are an interpretation by the Government Accountability Office (GAO, 2006) based in part on initial research by the staff of the New Orleans Times Picayune, and the values shown in Figure 2.7 have been added to this figure by our team, and are our own current best interpretation.

The situation is further clouded a bit, as the actual targeted levee and floodwall heights along a given section also varied slightly as a function of waterside topography, obstacles and vegetation, levee geometry, orientation and potential wind fetch (distance of potential wind travel across the top of open water), etc. as these would affect the potential run-up heights of storm waves. Variations for these types of issues were typically minor, on the order of two feet or less.

There is, however, no “typical” hurricane, nor associated storm surge, and the actual wind, wave and storm surge loadings imposed at any location within the overall flood protection system during an actual hurricane are a function of location relative to the storm, wind speed and direction, orientation of levees, local bodies of water, channel configurations, offshore contours, vegetative cover, etc. These loadings vary over time, as the storm moves progressively through the region.

Figures 2.8 and 2.9 show plots of storm surge levels resulting from numerical modeling simulations performed by the LSU Hurricane Research Center, for two different points in time during Hurricane Katrina, based on analyses of the storm track, wind speeds, regional topography and local conditions (marsh growth, soil stiffness, offshore contours, etc.) (Louisiana State University Hurricane Center, 2005.) The water levels shown in Figures 2.8 and 2.9 were predicted using a regionally calibrated numerical model, and the results shown in Figure 2.8 represent a point in time when the eye of the hurricane was first approaching the coast from the Gulf of Mexico, and those shown in Figure 2.9 correspond to a time when the eye of the storm was passing slightly to the east of New Orleans. These calculations are part of an overall single analysis of storm surge levels throughout the region, and throughout the continuous period of time as the storm approached and then passed through the region. Based on actual field observations and measurements of maximum storm surge levels at more than 100 locations throughout the region, this global analysis of storm surge levels is expected to be accurate (relative to surge levels that actually occurred) within approximately $\pm 15\%$ at all locations of interest for these current studies (IPET, 2006.)

Predicted and actual storm surge heights varied over time, at different locations, and the water levels shown in Figures 2.8 and 2.9 do not represent predictions of the peak storm surges noted at all locations. Instead, these images show calculated conditions at two interesting points in time when: (a) [Fig. 2.8] the initial large surge was being driven up against the coast of the Gulf of Mexico in the New Orleans region by the approaching storm, and (b) [Fig. 2.9] at a particularly critical moment when a large storm surge had first “inflated” (raised the level of) Lake Borgne, then the locally prevailing westward swirl of the counterclockwise hurricane winds threw the risen waters of Lake Borgne westward over the adjacent levees protecting eastern flanks of the New Orleans East and St. Bernard/Lower Ninth Ward protected areas, as shown schematically in Figure 2.11.

These types of storm surge modeling calculations are being performed by a number of research and investigation teams, and are constantly being calibrated and updated based on actual field measurements of high water marks, etc. The USACE’s IPET investigation team are devoting significant effort to these types of hydrodynamic analytical “hind-casts”, and the IPET back analyses provided to date to our UC Berkeley-led ILIT study team are in good agreement with the storm surge predictions shown in Figures 2.8 and 2.9 at most locations of interest for these studies (IPET; Draft Final Report, June 1, 2006).

Figure 2.10 shows an aggregate summary of the calculated peak storm surges, at any point in time during Hurricane Katrina, based on similar calculations performed by the IPET study (IPET; March, 2006). These calculations are very similar to those developed by the Louisiana investigation team, and both the IPET and Team Louisiana analyses will be used as a partial basis for estimation of storm surge levels and wave conditions in these current studies. The maximum flood stages calculated (predicted) by the two sets of analyses are generally in good agreement at

most points of interest. Agreement regarding storm waves is also generally good, but the differences between the two sets of predicted storm waves are a bit more significant at a few locations of interest. Discussions of the IPET and Team Louisiana hydrodynamic storm surge and storm wave calculations will be presented, in more detail, at locations of interest in the chapters that follow.

It should be noted that a number of different datums have been used as elevation references throughout the historic development of the New Orleans regional levee systems, and this situation is further complicated by ongoing subsidence in the region. This investigation has elected to resolve these differences between different datums, and to refer to all elevations in this report (as consistently as possible) in terms of elevation with respect to the NAVD88 (2004.65) datum; approximately “mean sea level” in the region. This particular version of the NAVD88 datum is currently thought to be within about 3-inches of Mean Sea Level (MSL) in the New Orleans region. For a more in-depth discussion of differences between the various datums used in the greater New Orleans region, please see IPET Interim Report No. 2 (IPET; March, 2006).

2.3.2 Overview of the Performance of the Regional Flood Protection System

Hurricane Katrina, as expected, produced a large onshore storm surge from the Gulf of Mexico. As shown in Figures 2.8 through 2.10 this produced significant overtopping of storm levees along the lower Mississippi River reaches in the Plaquemines Parish area, and numerous levee breaches occurred in this area, as shown previously in Figure 2.5. In simple terms, the “storm” levees of Plaquemines Parish were largely overwhelmed by the large storm surge; they were overtopped by the storm surge and by the large storm waves that accompanied the average rise (storm surge) in water levels. Fortunately, the Plaquemines Parish protected corridor is only sparsely populated, and the local inhabitants were acutely aware of the risk that they faced so that evacuation in advance of the storm was unusually complete.

Plaquemines Parish was largely inundated by the massive storm surge and the numerous resulting levee breaches. Most breaches appear to have been primarily the result of overtopping and erosion, and it is interesting to note that these breaches occurred mainly in the “storm” levees, while the “river” levees often better withstood the storm surge (and waves) without catastrophic erosion. The devastation within Plaquemines parish produced by this flooding was very severe, as described in Chapter 5. By approximately 7:00 a.m. on the morning of Monday, September 29, most of Plaquemines Parish was under water.

A more detailed discussion of the performance of the flood protection systems in the Plaquemines Parish area is presented in Chapter 5.

As the storm surge began to raise the water levels throughout the New Orleans region, it began to raise the water levels within the GIWW, MRGO and IHNC channels. As the water level within the IHNC began to rise, the first “breach” within the metropolitan New Orleans region (north of Plaquemines Parish) occurred at about 5:00 a.m. somewhere along the IHNC. This was evidenced by a pronounced, and short-lived, decrease in the rate of water level rise at two gage stations along the IHNC at this point in time. There are several breaches along this section of the IHNC that might have accounted for this observed water level gage behavior, and this is discussed in Chapter 8. This was a “non-catastrophic” failure; although the breach eroded and became enlarged by the flow, the “lip” of the breach remained above sea level. As a result,

although water flowed for a while into the protected area, this flow later stopped as the storm surge subsequently subsided. Simple calculations, based on flood stages and breach sequences and dimensions, suggest that less than 5% of the water that eventually flowed into the main Orleans East Bank (downtown) protected zone entered through this breach.

The large onshore storm surge also raised water levels within Lake Borgne (which is directly connected to the Gulf.) Lake Borgne rose up, and outgrew its normal banks. As the storm then passed to the east of New Orleans, the prevailing counterclockwise swirl of the storm winds drove the waters of Lake Borgne as a large storm surge to the west, against the eastern flank of the regional flood protection systems as shown schematically in Figure 2.11. This produced a storm surge estimated at approximately +16 to +18 feet (MSL), as shown in Figures 2.9 and 2.10.

This storm surge level exceeded the crest heights of the levees along a nearly 11-mile long stretch of the northeastern edge of the St. Bernard/Lower Ninth Ward protected area. The levees along this frontage were intended to be built to provide protection to a level of approximately +17.5 feet (MSL), but at the time of Hurricane Katrina many of the levees along this frontage had crest elevations approximately 2 to 4 feet lower than that. This was because the levees along this frontage had not yet been completed. These were “virgin” levees, being constructed on swampy foundation soils that had not previously had significant levees before. Accordingly, the swampy shallow foundation soils were both weak and compressible, and the levees were being constructed in stages to allow time for consolidation and settlement of the foundations soils. This process also allowed time for the drying of the very wet locally excavated soils used for some portions of the levee embankment fills, and also for increases in strength of the underlying foundation soils as they compressed under the weights of the growing levees.

Construction of the first phase of the levees along this frontage began in the late 1960’s. The last major work in this area prior to Katrina had been the construction of the third phase, in 1994-95. Since that time, the USACE had been waiting for Congressional appropriation of the funds necessary to construct the final stage (to the full design height, with allowance for anticipated future settlements.) Now it is too late.

In addition to the levees along this frontage being well below design grade, the manner of construction and the materials used were non-typical of most other USACE levees in the region. Ordinarily, the USACE requires the use of “cohesive” (clayey) soils to create an embankment fill that is both strong and relatively resistant to erosion. The levees along the “MRGO” frontage at the northeast edge of the St. Bernard Parish/Ninth Ward protected area were instead “sand core” levees (USACE, 1966). These levees were constructed using locally available soils, including dredge spoils from the excavation of the adjacent MRGO channel.

This is a region with predominantly marshy deposits, consisting largely of organic soils and soft paludal swamp clays with very high water contents. Beneath these generally poor surficial soils, the most common materials occurring at shallow, relatively accessible depths tend to be predominantly sandy soils that are highly erodeable and generally unsuitable for levee embankment fill. A decision was made, however, to attempt to use the locally available soils rather than importing higher quality soil fill materials. The USACE Design Memorandum describing this design refers to these as “sand core” levees (USACE, 1966).

The levees along this MRGO frontage section (along the northeastern edge of the St. Bernard protected area) were, in the end, constructed using large volumes of the spoil material excavated during the dredging of the adjacent MRGO shipping channel, and they contained unusually large quantities of highly erodeable sandy soils. In addition, some of the more cohesive (clayey) soils were too wet to be compacted effectively, and some sections of the embankments remained wet and soft for many years after construction. Chapter 6 presents a more detailed discussion of the erodeability of the levee embankments along the MRGO frontage. In simple terms, these levees were unusually massively erodeable, and this (combined with their lack of crest height) caused them to be unusually rapidly eroded as the storm surge from Lake Borgne approached and passed over, and through, these levees.

Based on analytical storm surge analyses and analytical “hindcasts” performed by various investigation teams, as well as eyewitness reports and timings of flooding and damages in St. Bernard Parish and the Ninth Ward, it is estimated that the storm surge passed over and through the MRGO levee frontage between approximately 6:00 to 7:00 a.m. The storm surge along the northeastern frontage of the St. Bernard Parish protected area peaked at approximately 7:30 to 8:00 a.m. (see Figure 2.9.) By the time the storm surge peaked along this important frontage, however, the unfinished “sand core” levees fronting Lake Borgne had been massively eroded and the brunt of the storm surge passed over and through the levees and raced across the undeveloped swamplands shown in Figure 2.11 towards the developed areas of St. Bernard Parish.

This is illustrated schematically in Figure 2.11. The levees along this frontage were so badly eroded, and so rapidly, that they did little to impede the passage of the storm surge which then crossed the roughly 7 to 10 miles of open swamp and reached the secondary levee that separates the northern (undeveloped) swampy section of this protected area from the populated southern section.

The secondary levee had not been intended to face the full fury of a storm surge of this magnitude; it had been assumed that the MRGO frontage levees would absorb much of the energy and provide more resistance. Accordingly, the storm surge passed over the secondary levee (which had lesser typical crest heights of only + 7.5 feet to + 10 feet, MSL) and washed into the populated regions of St. Bernard Parish. A number of minor breaches were produced by the overtopping (and erosion) of this secondary levee, but it is interesting to note that although this secondary levee must have been massively overtopped along much of its length, relatively little erosion damage resulted. The secondary levee was properly constructed, using compacted clayey soils, and the resulting levee embankment generally performed well with regard to resisting erosion. It was not, however, tall enough to restrain the massive overtopping from the storm surge which had passed so easily through the MRGO frontage levees.

The resulting carnage in St. Bernard Parish was devastating. A wall of water raced over the secondary levee; pushing homes laterally (Figure 2.16), flipping cars like toys and leaving them leaning against buildings, and driving large shrimp boats deep into the heart of residential neighborhoods (see Chapter 6.) The flooding of St. Bernard Parish was unexpectedly rapid. The peak depth of flooding in St. Bernard Parish was also unexpectedly deep because the floodwaters were pushed by the still rising storm surge (rather than having to flow more slowly, over time, through more finite breaches as the storm surge subsided; as occurred in most other parts of the greater New Orleans area) so that the top of the floodwaters at their peak within the developed areas were at an elevation well above mean sea level (approximately Elev. +12 feet, MSL.)

Indeed, after the storm surge subsided, “notches” were excavated through a number of local levees to let floodwaters drain under gravity loading from the significantly “plus mean sea level” flooding entrapped in some areas.

Figure 2.12 shows a plot of the locations where dead bodies were retrieved after the disaster as of December 2005. This map shows locations for only approximately 960 of the approximately 1,296 official deaths (to date) in the greater New Orleans area, but this map serves well to show the general distribution of deaths attributed to the flooding produced by this event. As shown in Figure 2.12, approximately 30% of these deaths occurred in St. Bernard Parish. In addition to those who perished, considerable damage was done to many thousands of homes and businesses in this area (see Chapter 6.)

The same storm surge from Lake Borgne that topped and eroded the levees along the “MRGO” frontage also pushed westward over the southeastern corner of the New Orleans East protected section, as shown in Figures 2.9 through 2.11, and this produced overtopping and a number of breaches, as shown previously in Figure 2.4. This was a principal source of the catastrophic flooding that subsequently made its way across the local undeveloped swamplands and into the populated areas of New Orleans East. Like the MRGO levee frontage discussed above, large portions of this levee frontage section had been constructed using materials excavated from the adjacent shipping channel (in this case the GIWW channel), and large portions of the levee were comprised of highly erodeable sandy and lightweight shell sand fill.

This storm surge from Lake Borgne also passed westward into a V-shaped “funnel” as it entered the shared GIWW/MRGO channel that separates the St. Bernard and New Orleans East protected areas, and this in turn resulted in an elevated surge of water that passed westward along the waterway to its juncture (at a “T”) with the IHNC channel, overtopping a number of levees and floodwalls on both the north and south sides of this east-west trending channel and producing levee distress and several breaches (as shown in Figures 2.4 and 2.11.) After reaching the “T” intersection with the IHNC channel, the surge then passed to the north and south (from the “T”) along the IHNC channel, periodically overtopping many (but not all) of the sections of levees and floodwalls lining the east and west sides of the IHNC, and causing a number breaches as shown in Figures 2.4 and 2.11. By about 6:45 to 7:00 a.m. overtopping (by up to as much as 1 to 2 feet at it’s peak at most locations) was occurring along a number of levee and floodwall sections lining the IHNC channel. This overtopping did not occur at all locations, and was only of limited duration (typically several hours or less) where it did occur.

A pair of major breaches occurred at the west end of the Lower Ninth Ward as this overtopping occurred along the IHNC, and the larger of these two breaches is shown (roughly seven weeks later, after construction of an interim repair embankment just outside the breach) in Figure 2.13. A large barge passed in through this breach, and can be seen in the rear of the photo. It is worth noting the tremendous scour-induced damage to the homes immediately inboard of this massive breach; most of the homes in Figure 2.13 were washed off of their foundations and transported laterally (often in pieces) by the inrushing floodwaters. A more detailed examination of the two large breaches at the west end of the Ninth Ward is presented in Chapter 6; Sections 6.4 and 6.5. The large breaches at the west end of the Lower Ninth Ward appear to have occurred by approximately 7:45 a.m. (Louisiana State University Hurricane Center, 2006.)

Like St. Bernard Parish, the breaches at the west end of the Lower Ninth Ward occurred before the storm surge peaked (at about 8:30 a.m. in the IHNC channel), so the Lower Ninth Ward was flooded to a level well above mean sea level before the storm surge subsequently subsided. This neighborhood, which had ground surface elevations of generally between about -3 to -6 feet (MSL) was flooded to elevations of up to as much as 10 to 12 feet above sea level. The resulting carnage, in terms of both loss of life (as shown in Figure 2.12) and destruction of homes and businesses was considerable, as the flooding rose above the tops of many of the one-story homes in this densely packed neighborhood.

The protected area of New Orleans East, directly to the north of the St. Bernard Parish/Ninth Ward protected area, had been breached at its southeastern corner by the initial storm surge and lateral rush from Lake Borgne (as shown schematically in Figure 2.11) by about 6:00 to 7:00 a.m., though the resulting breaches were confined to several locations so that the inflowing waters began to make their way across the undeveloped swamplands of the eastern portion of this protected area and timing is thus difficult to pin down with exactitude. The storm surge then passed laterally along the GIWW/MRGO east-west channel and produced another finite breach on the north side of this channel and several additional distressed sections. This breach added to the sources of water beginning to flow into this protected area.

The surge that passed west along the GIWW/MRGO east-west channel then pushed north along the IHNC, and produced several additional breaches and distressed sections, of varying severity, along the IHNC frontage as shown in Figure 2.4. These, too, added to the flow into the protected area of New Orleans East.

The lateral storm surge that passed westward along the east-west trending GIWW/MRGO channel between New Orleans East and St. Bernard Parish also attacked the west side of the IHNC channel, at the eastern edge of the main Orleans East Bank (downtown New Orleans) protected area. This produced three additional breaches along this frontage, as shown in Figures 2.4 and 2.11. Floodwaters began to flow into the main New Orleans metropolitan (downtown) protected area through these breaches between approximately 7:00 to 8:30 a.m. Although three of these breaches were relatively significant, all three breaches along this frontage failed to scour to significant depths. As a result, all three either had “lips” with lowest elevations above mean sea level, or there were points along the path from the IHNC to the breach that were above mean sea level. Accordingly, although all three breaches allowed some flow of water into the main Orleans East Bank (downtown) protected area, they allowed only limited flow and this flow stopped as the storm surge subsequently subsided. It would be the subsequent breaches in the drainage canals, to the northwest (along the edge of Lake Pontchartrain) that would prove to be devastating for this main (downtown) protected area.

As the hurricane then passed northwards to the east of New Orleans, the counterclockwise direction of the storm winds also produced a well-predicted storm surge southwards towards the south shore of Lake Pontchartrain. The lake level rose, but mainly stayed below the crests of most of the lakefront levees. The lake rose approximately to the tops of the lakefront levees at a number of locations, especially along the shoreline of New Orleans East, and there was moderate overtopping (or at least storm wave splash-over) and some resulting erosion on the crests and inboard faces of some lakefront levee sections along the Lake frontage. Significant overtopping occurred over a long section of concrete floodwall near the west end of the New Orleans East protected area lakefront (behind the Old Lakefront Airport), where the floodwall appears to have

been inexplicably lower than the adjacent earthen levee sections. This, too, added to the flow into the New Orleans East protected area, which was now continuing to fill with water even as the original storm surges subsided.

Farther to the west, the storm surge along the Pontchartrain lakefront (which peaked at about 9:00 to 9:30 a.m. at an elevation of about +10 feet, MSL) did not produce water levels sufficiently high as to overtop the crests of the concrete floodwalls atop the earthen levees lining the three drainage canals that extend from just north of downtown to Lake Pontchartrain; the 17th Street Canal, the Orleans Canal, and the London Avenue Canal. Three major breaches occurred along these canals, however, and these produced significant flooding of large areas within the Orleans East Bank protected area (as shown in Figure 2.4.) Figure 2.13 shows military helicopters lowering oversized bags of gravel into the levee breach on the east side of the 17th Street Canal, near the north end of the canal. Note that the flood waters have equilibrated, and that there is no net flow through the breach at the time of this photo.

The first breach along the drainage canals occurred near the south end of the London Avenue canal, between about 7:00 to 8:00 a.m. The second breach occurred near the north end of the London Avenue canal, and the best current estimates of the timing of this breach are between about 7:30 to 8:30 a.m. The third major breach occurred near the north end of the 17th Street canal. The main breach here occurred between about 9:00 to 9:15 a.m., but this may have been preceded by earlier visually observable distress at this same location. All three of these breaches rapidly scoured to depths well below mean sea level, so they continued to transmit water into the main Orleans East Bank (downtown) protected area after the storm surges subsided. A more detailed discussion and analyses of these catastrophic drainage canal breaches are presented in Chapter 8.

The resulting flooding of the main Orleans East Bank (Downtown) protected area was catastrophic, and resulted in at least 588 of the approximately 1,293 deaths attributed (to date) to the flooding of New Orleans by this event. Contributions to this flooding came from the overtopping and breaches along the IHNC channel at the east side of this protected area, but the majority of the flooding came from the three catastrophic failures along the drainage canals at the northern portion of this protected area.

In addition, one of the drainage canals (the Orleans Canal) had not yet been fully “sealed” at its southern end, so that floodwaters flowed freely into New Orleans during the storm surge through this unfinished drainage canal. A section of levee and floodwall approximately 200 feet in length had been omitted at the southern end of this drainage canal, so that despite the expense of constructing nearly 5 miles of levees and floodwalls lining the rest of this canal, as the floodwaters rose along the southern edge of lake Pontchartrain, the floodwaters did not rise fully within the Orleans canal; instead they simply flowed freely into downtown New Orleans.

Chapters 4 through 8 present a more detailed discussion of the performance of the flood protection systems nominally intended to protect the main Orleans East Bank area, and studies of the major failures and near failures within this critical area.

By approximately 9:30 a.m. the principal levee failures had occurred, and most of New Orleans was rapidly flooding.

2.3.3 Brief Comments on the Consequences of the Flooding of New Orleans

The consequences of the flooding of major portions of all four levee-protected areas of New Orleans were catastrophic. Approximately 85% of the metropolitan area of greater New Orleans was flooded, as shown in Figures 2.4 and 2.5. In Figure 2.4, the flooded areas are shown in pink, and those that remained still to be “unwatered” as late as September 28th are shown in darker blue. The blue cross-hatched areas were open, undeveloped swamplands, and these were also flooded but were not counted in determining the 85% flooding figure.

Large developed areas within all of the four main “protected areas” were flooded, and most remained inundated for two to three weeks before levee breaches could be repaired and the waters fully pumped out.

Figure 2.15 shows the approximate depth of flooding that remained on September 2nd, four days after Hurricane Katrina, in the St. Bernard Parish and Lower Ninth Ward protected area, based on an estimated surface water elevation of approximately +5 ft. (MSL) at that time. This is a significantly lower flood level than the estimated peak flooding to an elevation of up to +10 to 12 feet above mean sea level during the actual hurricane. The undeveloped swampland to the north of the populated areas can be seen in this Figure to also still be flooded on September 2nd, but the flood depths are not indicated.

Figure 2.16 shows the approximate depth of flooding that remained on September 2nd, again four days after the hurricane, in the New Orleans East protected area. As this protected area filled slowly during and after the hurricane, and as it was “unwatered” relatively slowly over the days and weeks that followed, this represents nearly the full depth of flooding in this area.

Figure 2.17 shows the approximate depth of flooding of the main Orleans East Bank (downtown) protected area on September 2nd. Like the New Orleans East protected area, this large protected “basin” filled relatively slowly over time. By September 2nd, the breaches had not yet all been closed by emergency repairs, so the depths of flooding in Figure 2.17 represent the nearly the full depth of flooding at its worst in this area.

Neighborhoods that were inundated exhibit stark evidence of this catastrophic flooding. Water marks, resembling oversized bathtub rings, line the sides of buildings and cars in these stricken neighborhoods, as shown in Figure 2.18. Household and commercial chemicals and solvents, as well as gasoline, mixed with the salty floodwaters in many neighborhoods, and at the time of this investigation’s first field visits shortly after the event the paint on cars below the watermarks on adjacent buildings had been severely damaged, and bushes and shrubs were browned below the watermarks, but often starkly green above. Driving through neighborhoods that had been flooded, there was often the impression that one was viewing a television screen where the color of the picture was somehow distorted or altered below a horizontal line; the level at which the floodwaters had been ponded. The devastation in these neighborhoods, and its lateral extent across many miles of developed neighborhoods, was stunning even to the many experienced members of our forensic teams that had seen numerous devastating earthquakes, tidal waves, and other major disasters.

Close to major breaches, the hydraulic forces of the inflowing floodwaters often had devastating effect on the communities. Figure 2.13 shows the devastation immediately inboard

from the large breach at the west end of the Ninth Ward site after the area had been unwatered. Note the numerous empty slabs where homes had been stripped away and scattered, mostly in pieces, across a large area.

Figure 2.19 shows another aspect of the flooding. This photograph shows a region within St. Bernard Parish in which some of the homes were transported from their original locations by the floodwaters, and then deposited in new locations. Figure 2.20 shows a number of homes in the Plaquemines Parish polder that were carried across the narrow polder (from left to right in this photograph) as the west side (left side of photo) “hurricane levee” or back levee was breached, and were then deposited on the crest of the Mississippi River levee. The water side slope face of the Mississippi River levee is clearly shown in this photograph, as evinced by the concrete slope face protection on the outboard side of the riverfront levee in the right foreground of the figure.

Figures 2.18 through 2.25 show examples of the devastation that occurred within the stricken flooded areas. The spray painted markings on the sides of the buildings in these areas were left by search and rescue teams, and they denote a number of important findings within each dwelling, including toxic contamination, etc. The most important numbers are those centered at the base of the large “X”, as these denote the number of dead bodies found within the building. In most cases this number was “0”, as for example in Figures 2.18 and 2.22. But this was not always the case. Figure 2.24 shows the outside of a dwelling in the Ninth Ward with a “3” beneath the X, indicating three deaths within. This was a housing unit, and the wheelchair ramp from the front door is askew at the bottom of the photograph. Figure 2.25 shows the muddy devastation, and a wheelchair, within this flooded structure.

Figure 2.26 gives another sense of perspective regarding the terrible and pervasive devastation wreaked by the flooding of large urbanized areas. This photo shows the flooding of an area of New Orleans East, but it could just as well be any of a number of large areas of New Orleans. Figure 2.27 gives a similar sense of perspective. In this photo, the flooded Lower Ninth Ward is in the foreground, and virtually every neighborhood shown (including those in the far background behind the tall downtown buildings) is flooded, excepting only the small area occupied by the tall buildings of the downtown area.

At the time of the writing of this report, the death toll from the flooding of New Orleans has risen to 1,293. It is expected to continue to climb a bit higher as some of those currently listed as “missing” will likely have been drawn out into the swamps and the Gulf by the floodwaters. Loss projections continue to evolve, but estimates of overall losses have now climbed to the \$100 to \$ 200 billion range for the metropolitan New Orleans region.

The members of this investigation team extend their hearts and their deepest condolences to those who were devastated by Hurricane Katrina, and by the flooding of most of New Orleans. The suffering and losses of those most intimately involved are almost beyond comprehension. It must be the goal and objective of all of us that a catastrophe of this sort never be allowed to happen again.

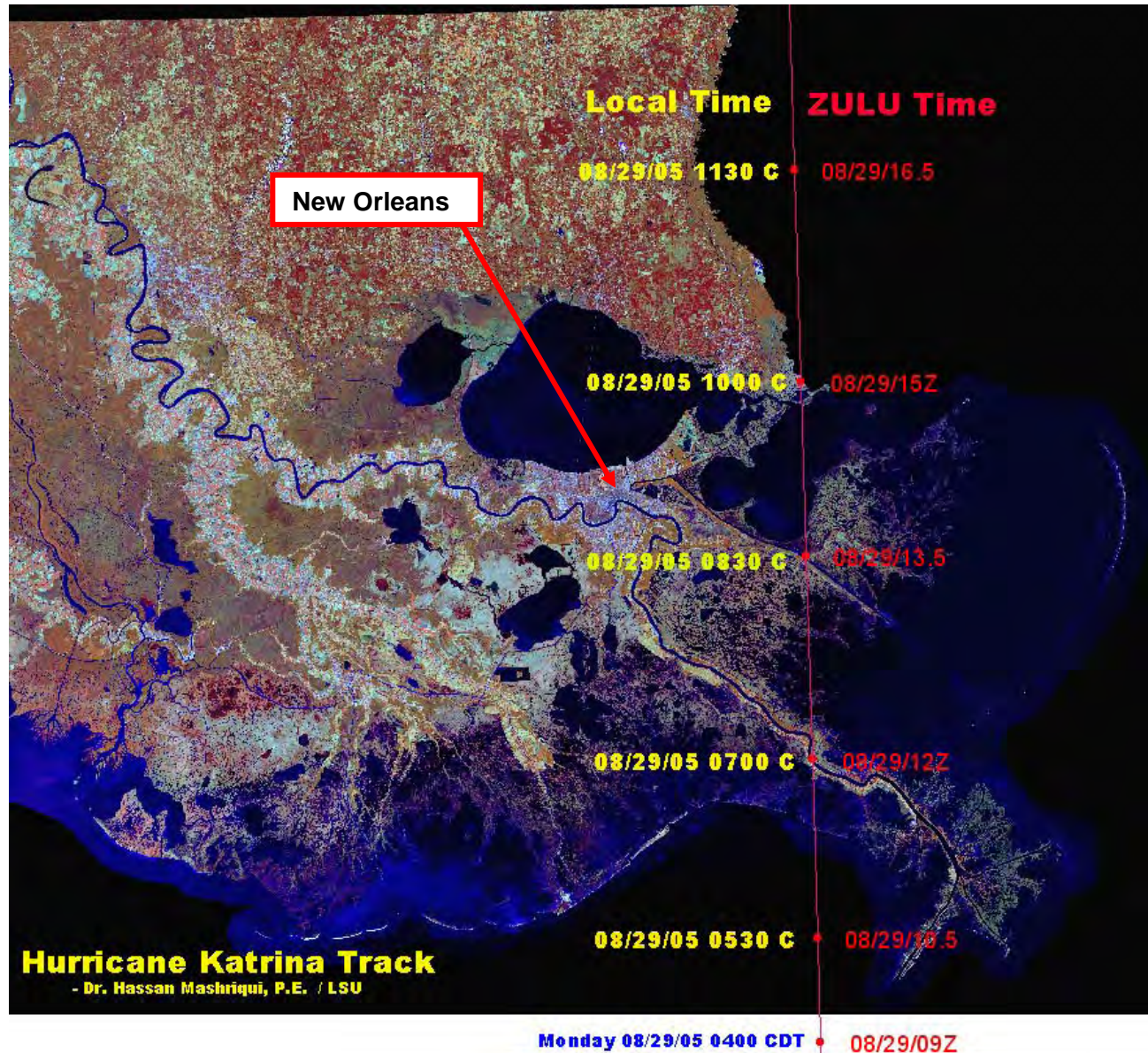
2.4 References

- Interagency Performance Evaluation Task Force, (2006), “Performance Evaluation, Status and Interim Results, Report 2 of a Series, Performance Evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System,” Interim Report No. 2, March 10, 2006.
- Interagency Performance Evaluation Task Force, (2006), “Performance Evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System”, Draft Final Report, June 1, 2006.
- Louisiana State University Hurricane Center, (2005), “Hurricane Katrina Advisory #22,” available from: <http://hurricane.lsu.edu/floodprediction/katrina22/>; date accessed: November 10, 2005.
- United States Army Corps of Engineers, (1967), “Lake Pontchartrain, LA, and Vicinity, Chalmette Area Plan, Hurricane Protection Levee First Lift, Sta. 594+00 – Sta. 770+00 (Not Continuous),” File No. H-8-24100, May 11, 1967.
- United States Army Corps of Engineers, (1966), “Lake Pontchartrain, LA. and Vicinity, Chalmette Area Plan, Design Memorandum No. 3, General Design,” November, 1966.



Source: <http://flhurricane.com/googlemap>

Figure 2.1: Location of New Orleans, and map of the path of the eye of Hurricane Katrina.



Source: Mashriqui, 2006

Figure 2.2: Traced path of the eye of Hurricane Katrina at landfall in the New Orleans area.

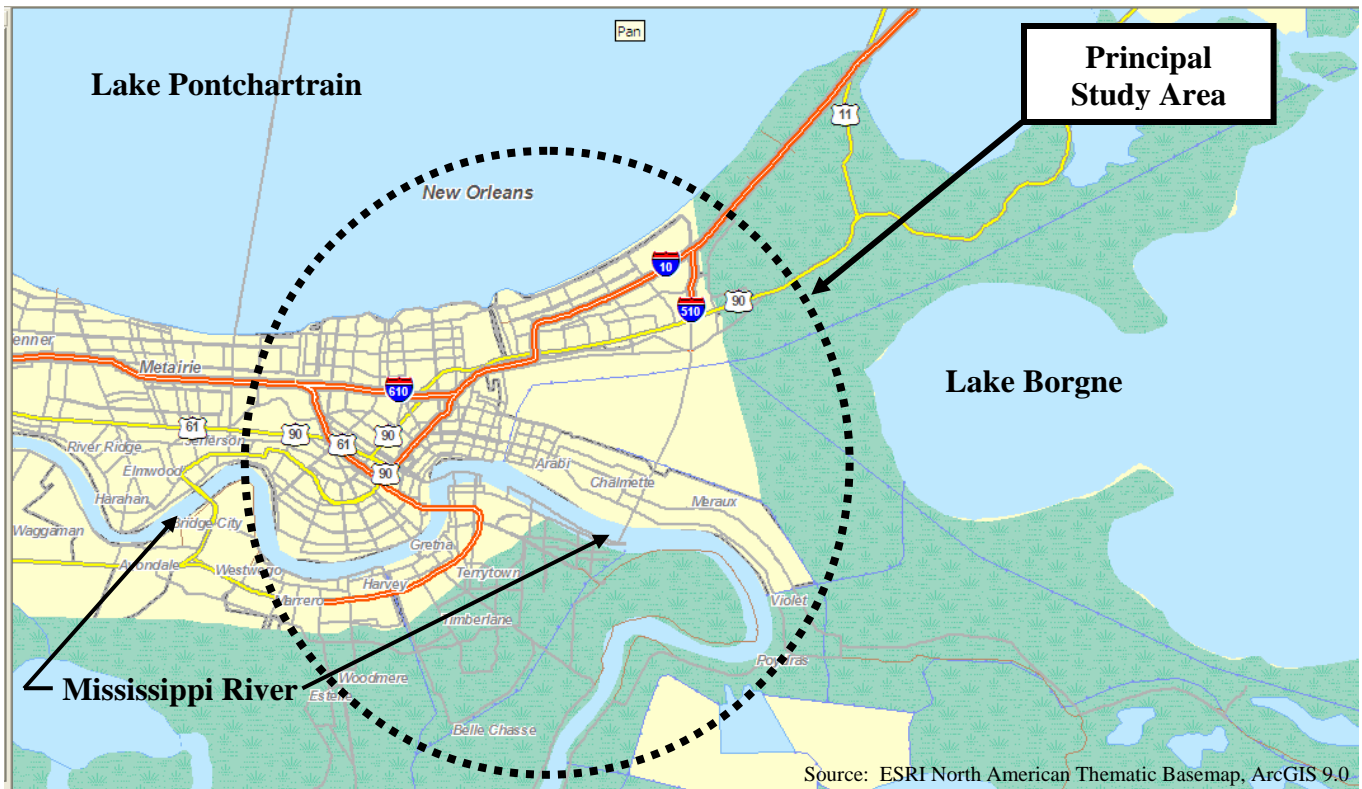
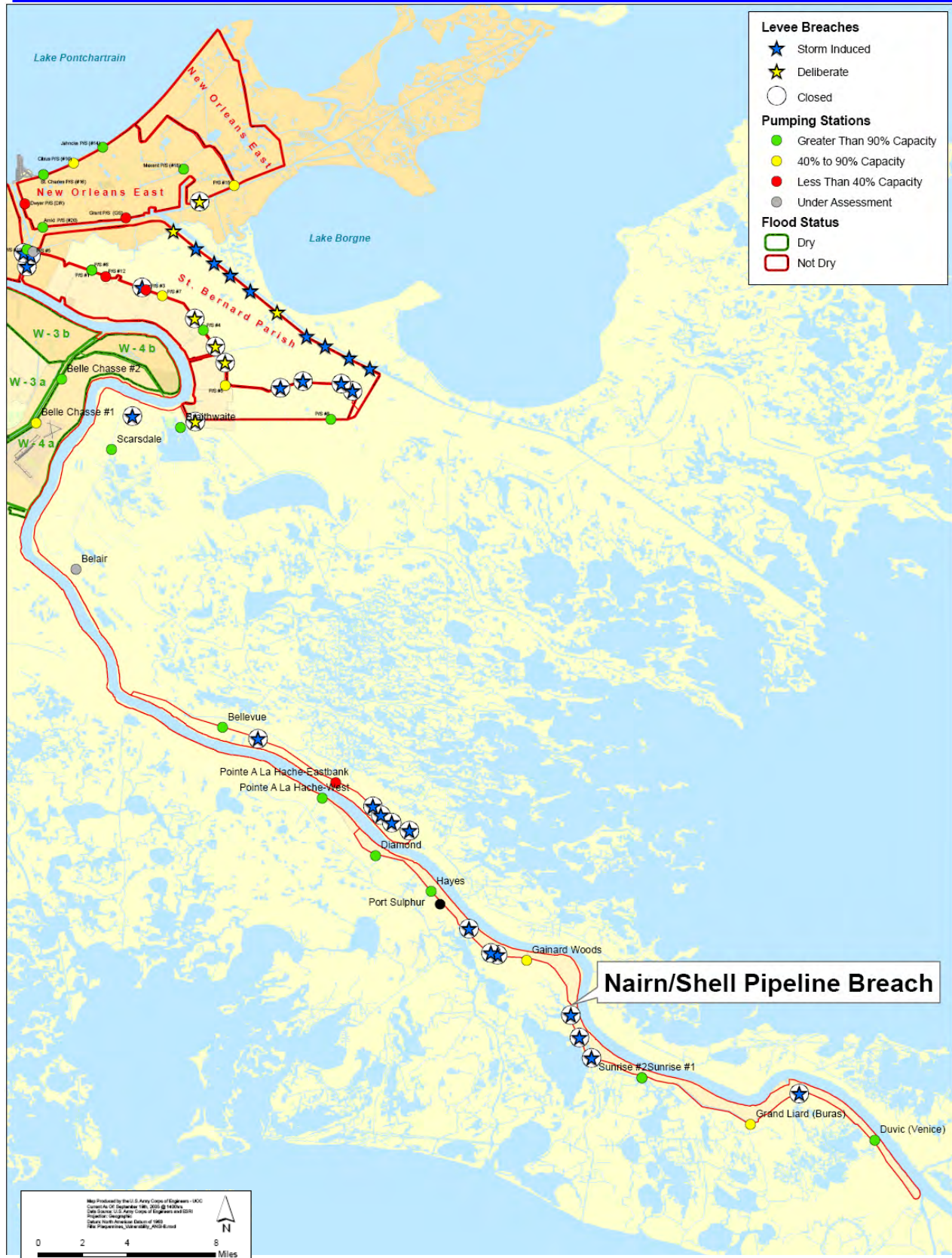


Figure 2.3: The greater New Orleans region levee and flood protection system Study Area.



Figure 2.4: Map showing principal features of the main flood protection rings or “protected areas” in the New Orleans area.



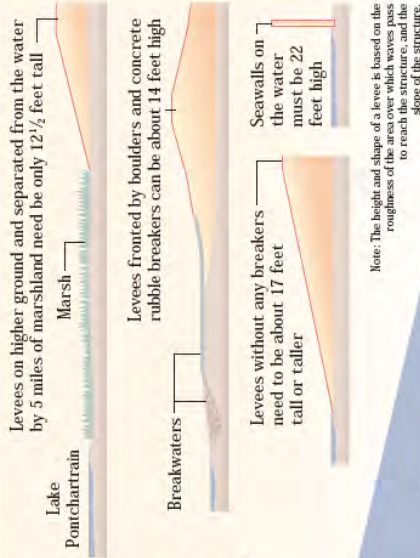
Source: Modified after USACE, 2005

Figure 2.5: Map showing the levee protected areas along the lower reaches of the Mississippi River (in the Plaquemines Parish Area.)

BARRIERS OF EARTH AND CONCRETE

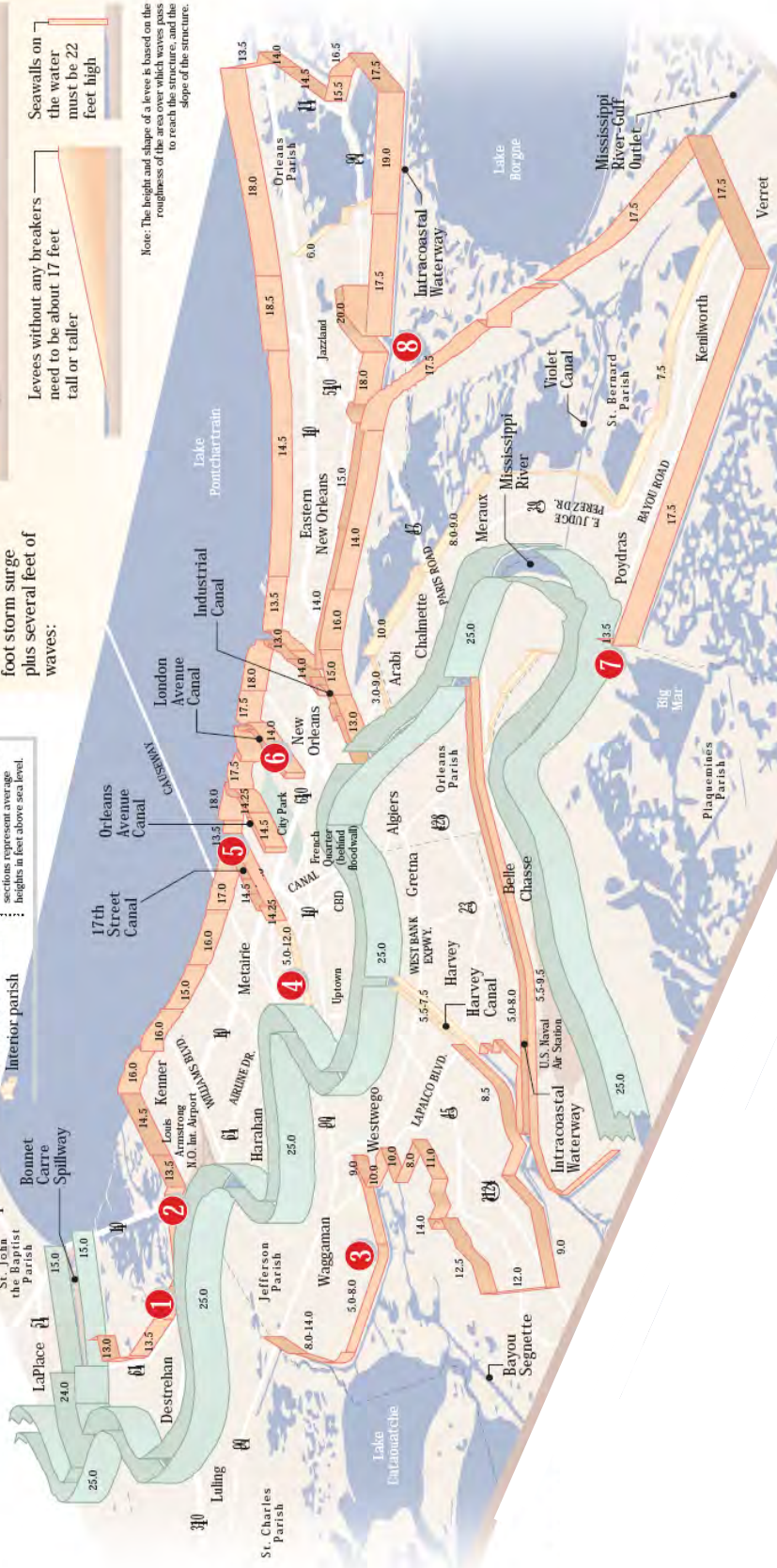
Levees and floodwalls that protect against flooding from both the Mississippi River and hurricanes are built by the Army Corps of Engineers and are maintained by local levee districts. The corps and the local districts share the construction cost of hurricane levees, while the Mississippi River levees are a federal project. Local levee districts also build and maintain nonfederal, lower-elevation levees with construction money from each district's share of property taxes and state financing.

HEIGHT ISN'T EVERYTHING
Different factors permit Lake Pontchartrain levees of varying elevations to withstand an 11 $\frac{1}{2}$ -foot storm surge plus several feet of waves:



LEVEES AND FLOODWALLS
Mississippi River
Hurricane protection
Interior parish

Notes: Levee and floodwall elevations are drawn with an extremely exaggerated vertical height but are in proportion to each other. The specific sections represent average heights in feet above sea level.

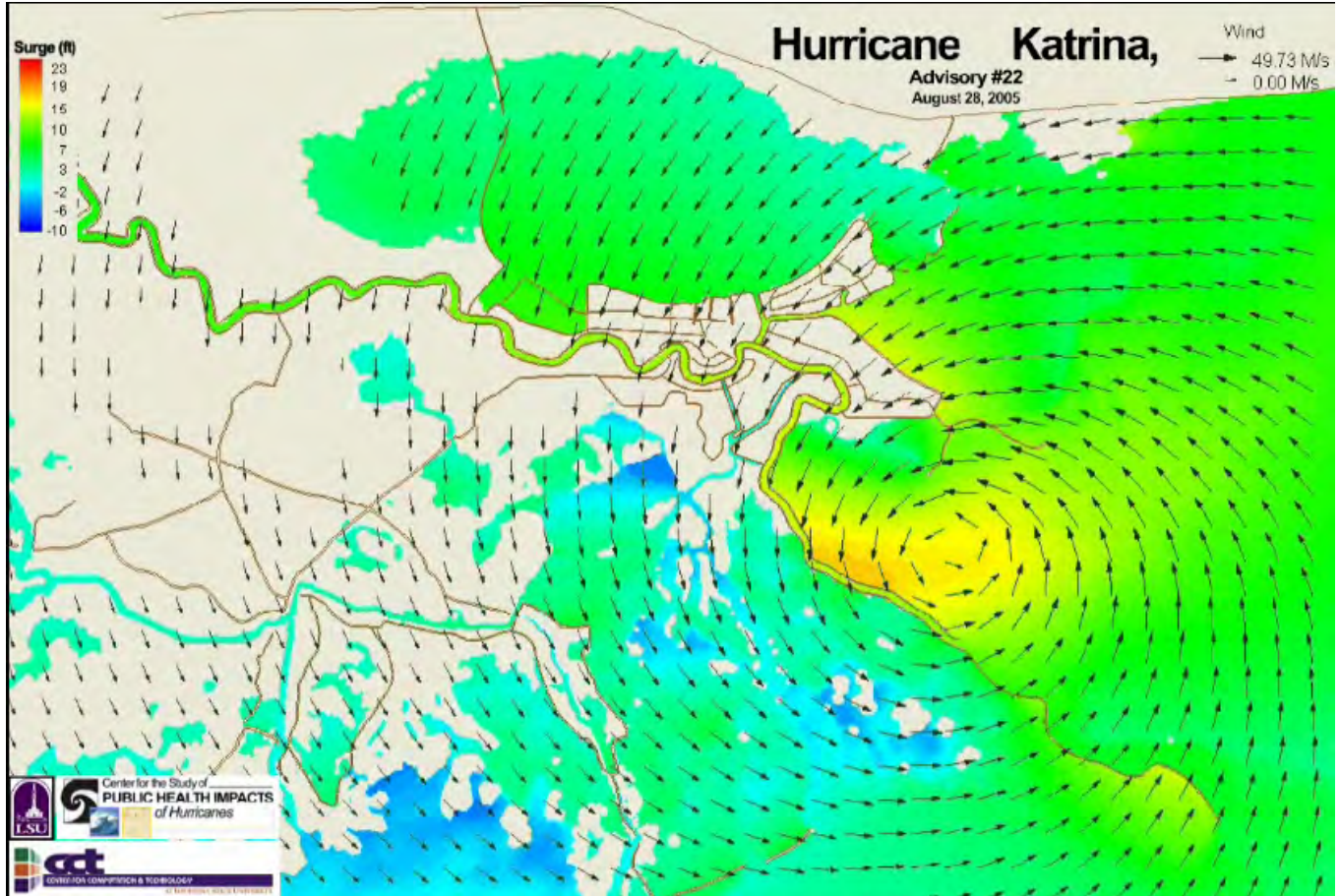


Source: Graphic by Emmet Mayer III/emayer@timespicayune.com (2005)

Figure 2.6: Map showing design flood stage elevations throughout the New Orleans region.

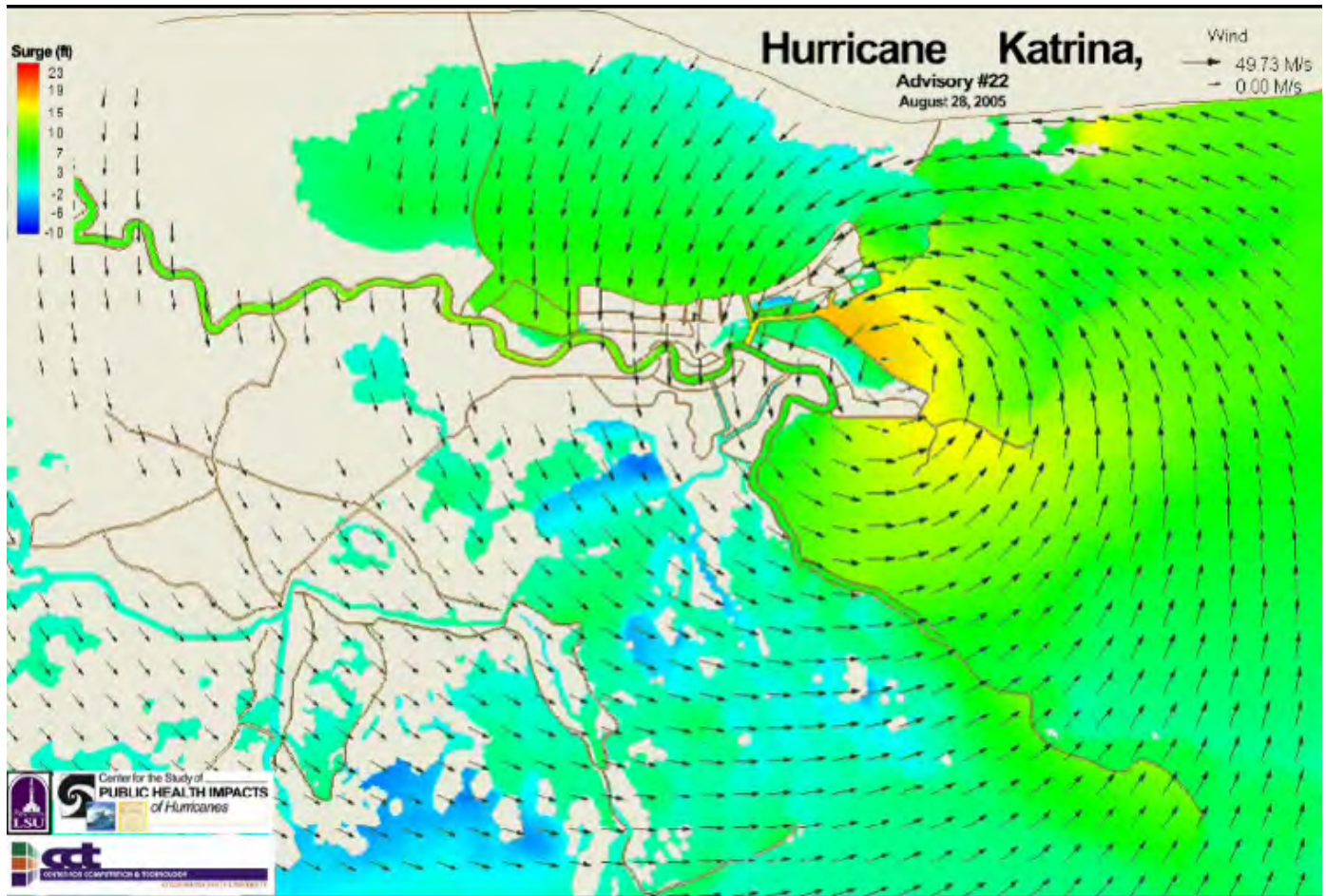


Figure 2.7: Map showing the design flood stage levels for selected locations in the New Orleans Area.



Source: <http://hurricane.lsu.edu/floodprediction/>

Figure 2.8: Calculated storm surge against the coast at about 7:30 am (CDT), August 29, 2006.



Source: <http://hurricane.lsu.edu/floodprediction/>

Figure 2.9: Map of calculated storm surge levels, at time when the eye of the storm passed close to the east of New Orleans at about 8:30 am (CDT).

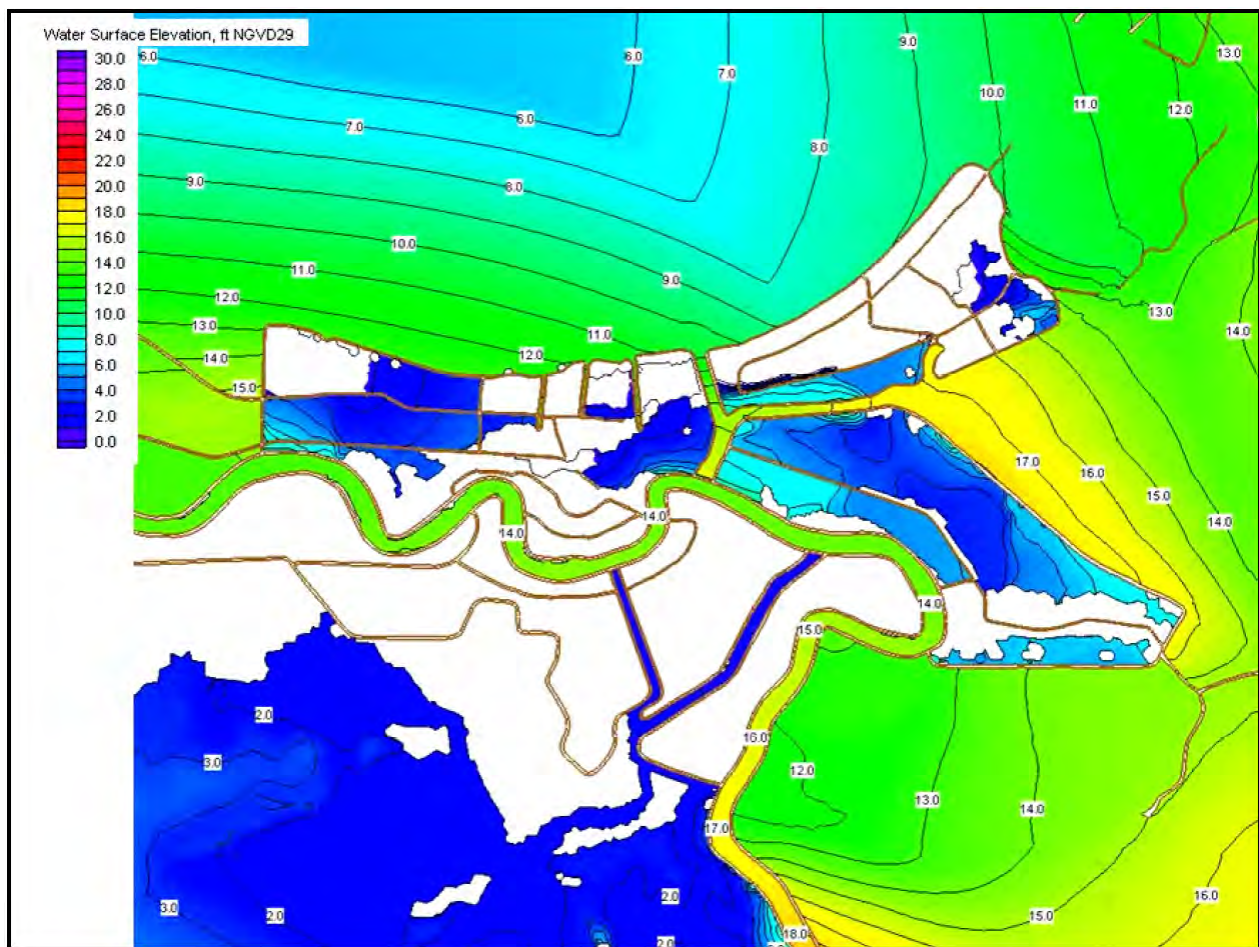


Figure V-37. Maximum computed storm surge using the ADCIRC model, metropolitan New Orleans vicinity (water levels in feet, NGVD 29)

Source: IPET Interim Report No. 2; April, 2006

Figure 2.10: Map showing calculated aggregate maximum storm surge levels (maximum values at any point in time).



Source: Modified after USACE, 2005

Figure 2.11: Storm surge overtopping the eastern flank of the regional flood protection system at the northeast edge of the St. Bernard Parish and Ninth Ward protected areas.

[illegible]

Figure 2.12: Map showing locations of confirmed deaths (as of December 2005) as a result of Hurricane Katrina.



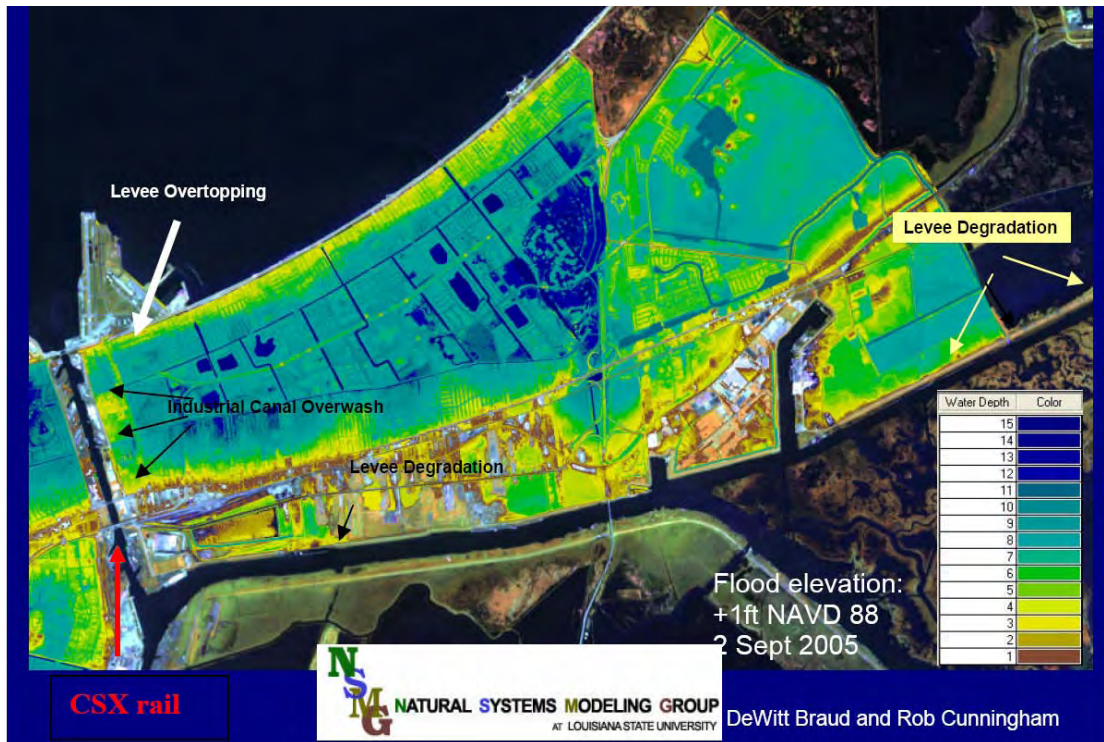
Photograph by Les Harder

Figure 2.13: Oblique view of the (south) levee break at the Inner Harbor Navigation Canal into the lower Ninth Ward.



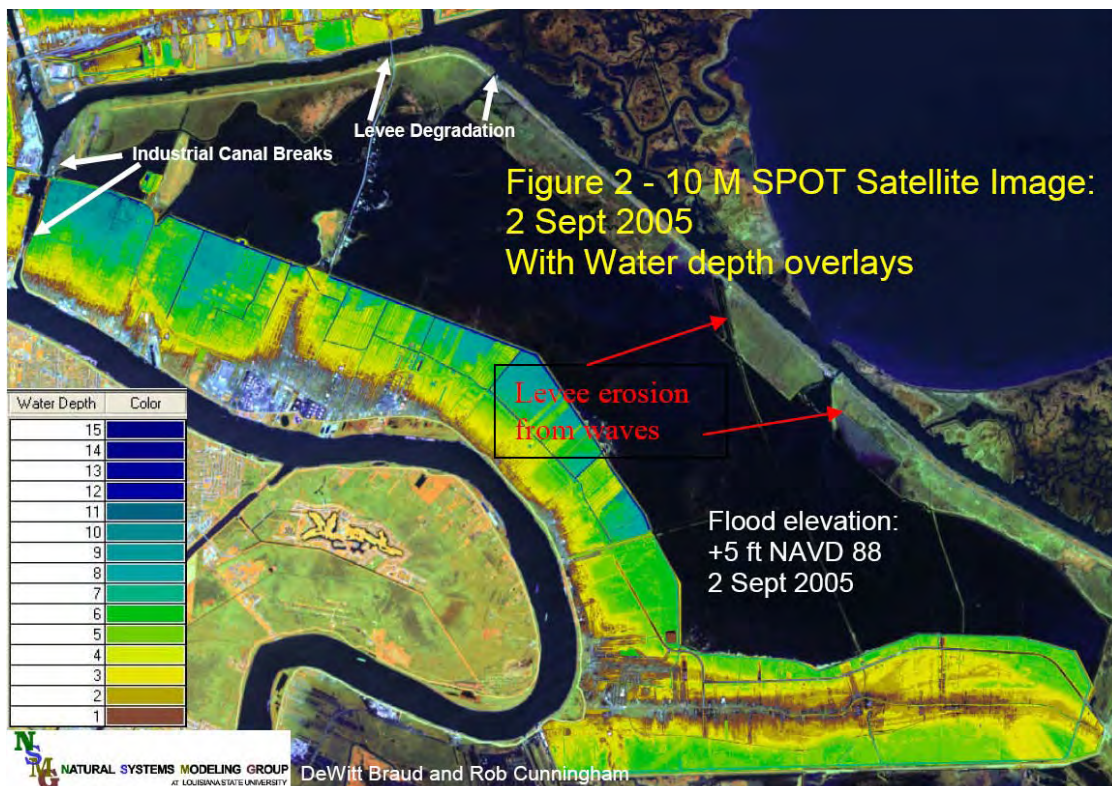
Photo courtesy of the U.S. Army Corps of Engineers

Figure 2.14: Initial closure of the large breach at the north end of the 17th Street Canal.



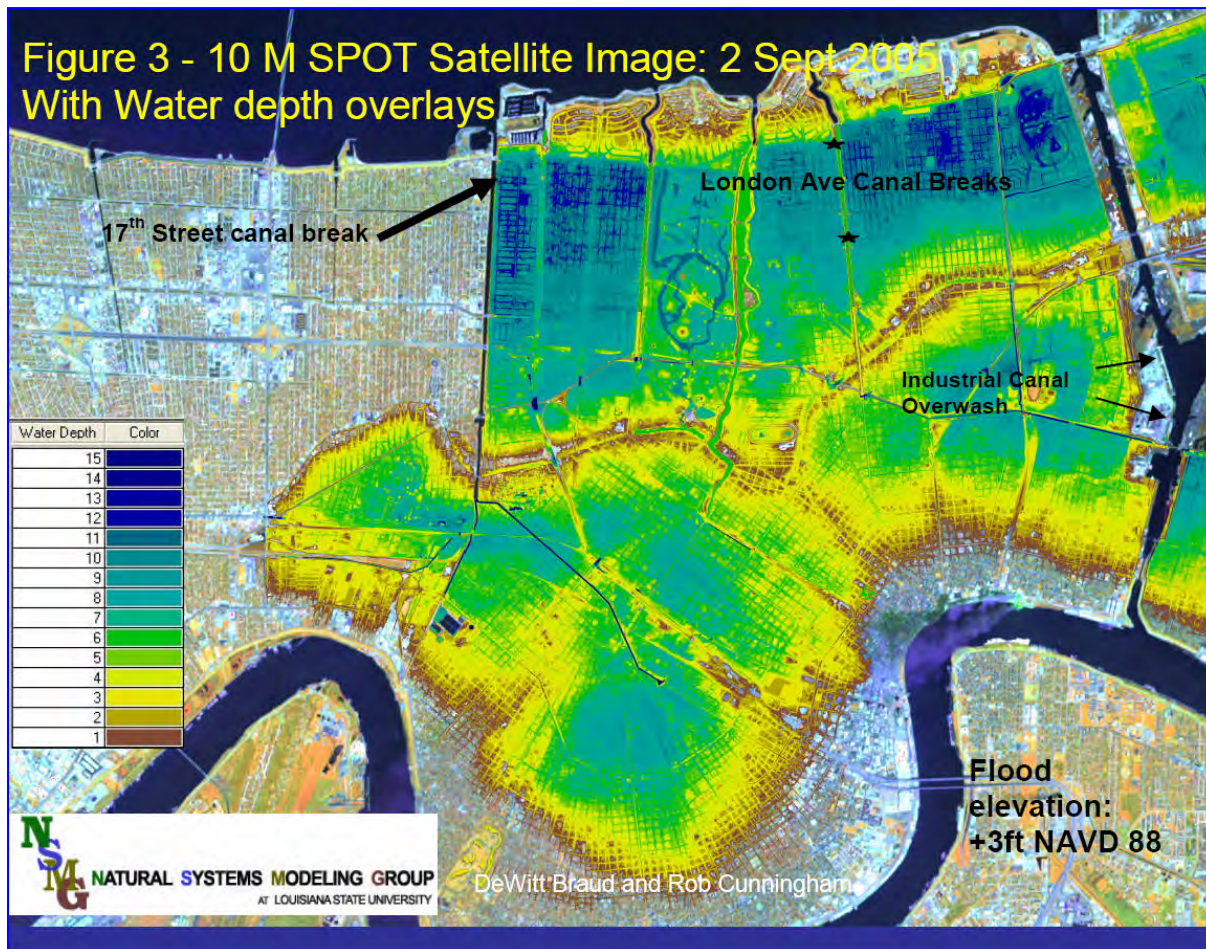
Source: LSU Hurricane Center, 2006

Figure 2.15: Depth of flooding of New Orleans East on September 2nd (4 days after Hurricane Katrina)



Source: LSU Hurricane Center, 2006

Figure 2.16: Depth of flooding of St. Bernard Parish and the Lower Ninth Ward on Sept. 2nd (4 days after Hurricane Katrina).



Source: LSU Hurricane Center, 2006

Figure 2.17: Depth of flooding of the Orleans East Bank (Downtown) protected area on September 2nd (4 days after Hurricane Katrina).



Photograph by Rune Storesund

Figure 2.18: High water marks remain on structures after temporary levee repairs have been completed and flood waters have been pumped out.



Photograph by Les Harder

Figure 2.19: Flooded neighborhood in St. Bernard Parish, showing homes floated off their foundations and transported by floodwaters.



Photograph by Les Harder

Figure 2.20: Homes in Plaquemines Parish carried from left to right in photo and strewn across the crown of the Mississippi Riverfront levee.



Photograph by Rune Storesund

Figure 2.21: Damage to a residential neighborhood in the 17th Street Canal area due to flooding.



Photograph by Rune Storesund

Figure 2.22: Search and rescue markings on a residence in the Canal District.



Photograph by Rune Storesund

Figure 2.23: Another view of flooding damage in the Canal District.



Photograph by Les Harder

Figure 2.24: Search and rescue team markings on a building in the lower Ninth Ward where three inhabitants died.



Photograph by Les Harder

Figure 2.25: View inside structure shown previously in Figure 2.21.



Photo Courtesy of http://www.wwltv.com/sharedcontent/breakingnews/slideshow/083005_dmnkatrina/7.html

Figure 2.26: Neighborhood in New Orleans East fully flooded.



Photo courtesy of http://www.wwltv.com/sharedcontent/breakingnews/slideshow/083005_dmnkatrina/7.html

Figure 2.27: View of the City of New Orleans at the peak of the flooding.

CHAPTER THREE: GEOLOGY OF THE NEW ORLEANS REGION

3.1 General Overview of the Geology of New Orleans

3.1.1 Introduction

Hurricane Katrina brought devastation to New Orleans and the surrounding Gulf Coast Region during late August 2005. Although there was wind damage in New Orleans, most of the devastation was caused by flooding after the levee system adjacent to Lake Pontchartrain, Lake Borgne and Inner Harbor areas of the city systematically failed. The storm surge fed by winds from Hurricane Katrina moved into Lake Pontchartrain from the Gulf of Mexico through Lake Borgne, backing up water into the drainage and navigation canals serving New Orleans. The storm surge overwhelmed levees surrounding these engineered works, flooding approximately 80% of New Orleans.

Although some levees/levee walls were overtopped by the storm surge, the London Avenue and 17th Street drainage canal walls were not overtopped. They appear to have suffered foundation failures when water rose no higher than about 4 to 5 feet below the crest of the flood walls. This occurrence has led investigators to carefully investigate and characterize the foundation conditions beneath the levees that failed. A partnership between the U.S. Geological Survey's Mid-Continent Geologic Science Center and the University of Missouri – Rolla, both located in Rolla, MO, was established in the days immediately after the disaster to make a field reconnaissance to record perishable data. This engineering geology team was subsequently absorbed into the forensic investigation team from the University of California, Berkeley, funded by the National Science Foundation.

The team has taken multiple trips to the devastated areas. During these trips team members collected physical data on the levee failures, much of which was subsequently destroyed or covered by emergency repair operations on the levees. Our team also logged a series of subsurface exploratory borings to characterize the geological conditions present in and around the levee failure sites.

3.1.2 Evolution of the Mississippi Delta beneath New Orleans

The Mississippi River drains approximately 41% of the Continental United States, a land area of 1.2 million mi² (3.2 million km²). The great majority of its bed load is deposited as subaerial sediment on a well developed flood plain upstream of Baton Rouge, as opposed to subaqueous deposits in the Gulf of Mexico. The Mississippi Delta has been laid down by an intricate system of distributary channels; that periodically overflow into shallow swamps and marshes lying between the channels (Figure 3.1, upper). The modern delta extends more or less from the present-day position of Baton Rouge (on the Mississippi River) and Krotz Springs (on the Atchafalaya River). The major depositional lobes are shown in Figure 3.1 (lower).

Between 12,000 to 6,000 years ago sea level rose dramatically as the climate changed and became warmer, entering the present interglacial period, which geologists term the

Holocene Epoch (last 11,000 years). During this interim, sea level rose approximately 350 feet, causing the Gulf of Mexico to retreat into southeastern Louisiana inundating vast tracts of coastline. By 7,000 years ago sea level had risen to within about 30 feet of its present level. By 6,000 years ago the Gulf had risen to within 10 to 15 feet of its present level.

The modern Mississippi Delta is a system of distributary channels that have deposited large quantities of sediment over the past 6,000 to 7,000 years (Figure 3.1 –upper). Six major depositional lobes, or coalescing zones of deposition, have been identified, as presented in Figure 3.1 (lower). In southeastern Louisiana deltaic sedimentation did not begin until just the last 5,000 years (Saucier, 1994). Four of these emanate from the modern Mississippi River and two from the Atchafalaya River, where the sediments reach their greatest thickness. The St. Bernard Delta extending beneath Lake Borgne, Chandeleur and Breton Sounds to the Chandeleur and Breton Shoals was likely deposited between 600 and 4,700 years ago. The 50+ miles of the modern Plaquemines-Balize Delta downstream of New Orleans has all been deposited in just the last 800 to 1,000 years (Darut et al. (2005).

During this same period (last 7,000 years) the Mississippi River has advanced its mouth approximately 200 river miles into the Gulf of Mexico. The emplacement of jetties at the river's mouth in the late 1870's served to accelerate the seaward extension of the main distributary passes (utilized as shipping channels) to an average advance of about 70 meters per year, or about six times the historic rate (Coleman, 1988; Gould, 1970). The combination of channel extension and sea level rise has served to flatten the grade of the river and its adjoining flood plains, diminishing the mean grain size of the river's bed load, causing it to deposit increasing fine grained sediments. Channel sands are laterally restricted to the main stem channel of the Mississippi River, or major distributary channels, or "passes", like the Metairie-Gentilly Ridge. The vast majority of the coastal lowland is infilled with silt, clay, peat, and organic matter.

Geologic sections through the Mississippi Embayment show that an enormous thickness of sediment has been deposited in southern Louisiana (Figure 3.2). During the Quaternary Period, or Ice Ages, (11,000 to 1.6 million years ago) the proto Mississippi River conveyed a significantly greater volume of water on a much steeper hydraulic grade. This allowed large quantities of graveliferous deposits beneath what is now New Orleans, reaching thicknesses of up to 3600 feet (Figures 3.2 and 3.3). These stiff undifferentiated Pleistocene sands and gravels generally lie between 40 and 150 feet beneath New Orleans, and much shallower beneath Lake Pontchartrain and Lake Borgne (as one approaches the Pleistocene outcrop along the North Shore of Lake Pontchartrain).

Just south of the Louisiana coast, the Mississippi River sediments reach thicknesses of 30,000 feet or more. The enormous weight of this sediment mass has caused the earth's crust to sag in this area, resulting in a structure known as the Gulf Geosyncline (Figure 3.2). Flow of mantle material from below the Gulf Geosyncline is causing an uplift along about the latitude of Wiggins, MS. This is one cause of subsidence in South Louisiana (discussed in Section 3.7.2).

Figure 3.4 presents a generalized geologic map of the New Orleans area, highlighting the salient depositional features. Depth contours on the upper Pleistocene age (late Wisconsin glacial stage) horizons are shown in red. Sea level was about 100 feet lower than present

about 9000 years ago, so the -100 ft contour represents the approximate shoreline of the Gulf at that time, just south of the current Mississippi River channel. Figure 3.5 presents a more detailed view of the dissected late Wisconsin stage erosional surface beneath New Orleans. This system emanates from the Lake Pontchartrain depression and reaches depths of 150 feet below sea level where it is truncated by the modern channel of the Mississippi River, which is not as deeply incised. A veneer of interdistributary deltaic deposits covers this older surface and is widely recognized for having spawned differential settlement of the cover materials where variations in thickness are severe, such as the Garden District.

3.1.3 Pine Island Beach Trend

Relict beach deposits emanating from the Pearl River are shown in stippled yellow on Figure 3.4. Saucier (1963) named these relic beaches the Pine Island and Miltons Island beach trends. These sands emanate from the Pearl River between Louisiana and Mississippi, to the northeast. The Miltons Island Beach Trend lies beneath the north shore of Lake Pontchartrain, while the Pine Island Beach Trend runs northeasterly, beneath the Lakeview and Gentilly neighborhoods of New Orleans up to the Rigolets. The Pine Island Beach Trend is believed to have been deposited when sea level had almost risen to its present level, about 4500 years ago. At that juncture, the rate of sea level rise began to slow and there was an unusually large amount of sand being deposited near the ancient shoreline by the Pearl River, which was spread westerly by longshore drift, in a long linear sand shoal, which soon emerged into a beach ridge along a northeast-southwest trend (Saucier, 1963). The subsequent development of accretion ridges indicate that shoreline retreat halted and the beach prograded southwestward, into what is now the Gentilly and Lakeview areas. By about 5,000 years ago, the beach has risen sufficiently to form a true barrier spit anchored to the mainland near the present Rigolets, with a large lagoon forming on its northern side (what is now Lake Pontchartrain, which occupies an area of 635 mi²).

Sometime after this spit formed, distributaries of the Mississippi River (shown as yellow bands on Figure 3.4) began depositing deltaic sediments seaward of the beach trend, isolating it from the Gulf of Mexico. The Pine Island Beach Trend was subsequently surrounded and buried by sediment and the Pine Island sands have subsided 25 to 45 feet over the past 5,000 years (assuming it once stood 5 to 10 feet above sea level). The distribution of the Pine Island Beach Trend across lower New Orleans is shown in Figure 3.6. The Pine Island sands reach thicknesses of more than 40 feet in the Gentilly area, but diminish towards the Lakeview area, pinching out near the New Orleans/Jefferson Parish boundary (close to the 17th Street Canal breach). The Pine Island beach sands created a natural border that helped form the southern shoreline of Lake Pontchartrain, along with deposition by the Mississippi River near its present course. Lake Pontchartrain was not sealed off entirely until about 3,000 years ago, by deposition in the St. Bernard's Deltaic lobe (Kolb, Smith, and Silva, 1975). The Pine Island Beach Trend tapers out beneath Jefferson Parish, as shown in Figures 3.4 and 3.6.

3.1.4 Interdistributary Zones

Most of New Orleans' residential areas lie within what is called an interdistributary zone, underlain by lacustrine, swamp, and marsh deposits, shown schematically in Figure 3.7.

This low lying area rests on a relatively thin deltaic plain, filled with marsh, swamp, and lacustrine sediments. The drainage canals were originally constructed between 1833-78 on intertributary embayments, which are underlain by fat clays deposited in a quiet water, or paludal, environment (Kolb and Van Lopik, 1958).

Intertributary sediments are deposited in low lying areas between modern tributary channels and old deltas of the Mississippi River, shown schematically in Figure 3.8. The low angle bifurcation of tributary streams promotes trough-like deposits that widen towards the gulfward. Sediment charged water spilling over natural channel levees tends to drop its coarse sediment closest to the channel (e.g. Metairie and Gentilly Ridges) while the finest sediment settles out in shallow basins between the distributaries. Fine-grained sediment can also be carried into the intertributary basins through crevasse-splays well upstream, which find their way into low lying areas downstream. Storms can blow sediment-laden waters back upstream into basins, while hurricanes can dump sediment-laden waters onshore, though these may be deposited in a temporarily brackish environment.

Considerable thickness of intertributary clays can be deposited as the delta builds seaward. Kolb and Van Lopik (1958) noted that intertributary clays often grade downward into prodelta clays and upward into richly organic clays of swamp or marsh deposits. The demarcation between clays deposited in these respective environments is often indistinct. True swamp or marsh deposits only initiate when the water depth shallows sufficiently to support vegetation (e.g. cypress swamp or grassy marsh). The intertributary zone is typified by organic clays, with about 60% by volume being inorganic fat clays, and 10% or less being silt (usually in thin, hardly discernable stringers). Kolb and Van Lopik (1958) reported cohesive strengths of intertributary clays as ordinarily being something between 100 and 400 psf. These strengths, of course, depend also on the past effective overburden pressure.

Careful logging is required to identify the depositional boundary between intertributary (marsh and swamp) and prodelta clays (Figure 3.9). The silt and fine sand fractions in intertributary materials are usually paper-thin partings. Prodelta clays are typified by a massive, homogeneous appearance with no visible planes or partings. Geologically recent intertributary clays, like those in lower New Orleans, also tend to exhibit underconsolidation, because they were deposited so recently. Intertributary clays in vicinity of South Pass (45 miles downstream of New Orleans) exhibit little increase in strengths to depths of as much as 375 ft. This is because these materials were deposited rapidly, during the past 600 to 1,000 years, and insufficient time has passed to allow for normal consolidation, given the low drainage characteristics of the units. This phenomenon was noted and analyzed for offshore clays by Terzaghi (1956). The older prodelta clays underlying recent intertributary clays tend to exhibit almost linear increase of density and strength with depth, because these materials were deposited very slowly. So, the environment of deposition greatly impacts soil strength.

3.1.5 Paludal environments

Paludal environments on the Mississippi River deltaic plain are characterized by organic to highly organic sediments deposited in swamps and marshes. Paludal environments

are typified half-land and half-water, with water depths seldom exceeding two feet above mean gulf level. 90% of New Orleans is covered by swamp or marsh deposits (excluding filled areas). Lacustrine (lake) and tidal channel deposits can be complexly intermingled with swamp and marsh deposits.

3.1.5.1 Marshes

More than half of the New Orleans area was once covered by marshes, essentially flat areas where the only vegetation is grasses and sedges. Tufts of marsh grass often grow with mud or open water between them. When these expanses are dry, locals often refer to them as “prairies.” As the marshes subside, grasses become increasingly sensitive to increasing salinity. As grasses requiring fresh water die out, these zones transition into a myriad of small lakes, eventually becoming connected to an intricate network of intertidal channels that rise and fall with diurnal tides. These are often noted on older maps as “brackish” or “sea marshes” to discern them from adjoining fresh water swamps and marshes (Figure 3.9).

Marsh deposits in New Orleans are typically comprised of organic materials in varying degrees of decomposition. These include peats, organic oozes, and humus formed as marsh plants die and are covered by water. Because the land is sinking, subaerial oxidation is limited, decay being largely fomented by anerobic bacteria. In stagnant water thick deposits consisting almost entirely of organic debris are commonplace. The low relative density of these materials and flooded nature provides insufficient effective stress to cause consolidation. As a consequence, the coastal marsh surface tends to “build down,” as new vegetation springs up each year at a near-constant elevation, while the land continues to subside. In areas bereft of inorganic sediment, thick sequences of organic peat will accumulate, with low relative density. If the vegetation cannot keep pace with subsidence, marine waters will inundate the coastal marsh zone, as noted in the 1849 map in Figure 3.10.

Peats are the most common variety of marsh deposits in New Orleans. They usually consist of brown to black fibrous or felty masses of partially decomposed vegetative matter. Materials noted on many of the older boring logs as “muck” or “swamp muck” are usually detrital organic particles transported by marsh drainage or decomposed vegetative matter. These mucks are watery oozes that exhibit very low shear strength and cannot support any appreciable weight.

Inorganic sediments may also accumulate in marshes, depending on the nearness of a sediment source(s). Common examples are sediment-laden marine waters and muddy fluvatile waters. Brackish marsh deposits interfinger with fresh water deposits along the southern shore of Lake Pontchartrain, but dominate the shoreline around Lake Borgne. Floating marsh materials underlie much of the zone along old watercourses, like Bayou St. John and Bayou des Chapitoulas. Kolb and Van Lopik (1958) delineated four principal types of marsh deposits in New Orleans:

1. Fresh water marsh consists of a vegetative mat underlain by clays and organic clays. Fresh water marshes generally form as a band along the landward border of established marshes and in those areas repeatedly subjected to fresh water inundation. In most instances an upper mat of roots and plant parts at least 12 inches thick overlies fairly soft organic clays,

which become firmer and less organic with depth. Peat layers are often discontinuous and their organic content is usually between 20 and 50%.

2. Floating marsh or flotant is a vegetative mat underlain by organic ooze. This is sometimes referred to as a “floating fresh marsh” or “floating three-cornered grass marsh.” The vegetative mat is typically between 4 and 14 inches thick, floating on 3 to 15 ft of finely divided muck or organic ooze, grading into clay with depth. The ooze often consolidates with depth and grades into a black organic clay or peat layer.

3. Brackish-fresh water marsh sequence consists of a vegetative mat underlain by peat. The upper mat of roots and recent marsh vegetation is typically 4 to 8 inches thick and underlain by 1 to 10 ft of coarse to medium textured fibrous peat. This layer is often underlain by a fairly firm, blue-grey clay and silty clay with thick lenses of dark grey clays and silty clays with high organic contents. The great majority of marsh deposits in New Orleans are of this type, with a very high peat and humus content, easily revealed by gravimetric water content and/or dry bulk density values.

4. Saline-brackish water marsh is identified by a vegetative mat underlain by clays. These are sometimes termed “drained salt marshes” on older maps. The typical sequence consists of a mat of roots, stems, and leaves from 2 to 8 inches thick, underlain by a fairly firm blue-grey clay containing roots and plant parts. Tiny organic flakes and particles are disseminated through the clay horizon. The clays tend to become less organic and firmer with depth. The saline to brackish water marsh occupies a belt ½ to 8 miles wide flanking the present day shoreline, along the coast.

The strengths of marsh deposits are generally quite low, depending on their water content. Embankments have been placed on vegetative mats underlain by ooze, supporting as much as 2 or 3 psi of loading, provided it is uniformly applied over reasonable distances, carefully (Kolb and Van Lopik, 1958). Field observations of sloped levees founded on such materials indicate failure at heights of around 6 feet, which exert pressures close to those cited above.

3.1.5.2 Swamps

Before development, swamps in the New Orleans were easily distinguishable from marshes because of the dense growth of cypress trees. All of the pre-1900 maps make reference to extensive cypress marshes in lower New Orleans, between the French Quarter and Lake Pontchartrain (Figure 3.11). Encountering cypress wood in boreholes or excavations is generally indicative of a swamp environment. These cypress swamps thrived in 2 to 6 feet of water, but cannot regenerate unless new influx of sediment is deposited in the swamp, reducing the water depth. Brackish water intrusion can also cause flocculation of clay and premature die out of the cypress trees.

Two layers of cypress swamp deposits are recognized to extend over large tracts of New Orleans (WPA-LA, 1937). The upper layer is the historic swamp occupying the original ground surface where infilling has occurred since the founding of the city in 1718; and the second; is a pervasive layer of cypress tree stumps that lies 20 to 30 feet below the ground surface, around -25 ft MGL (Mean Gulf Level). This older cypress forest was undoubtedly

killed off and buried in a significant pre-historic flood event, fomented by considerable deposition of inorganic sediment. This sudden influx of sediment may have come from a crevasse-splay along the Mississippi River upstream of New Orleans, as in most of the damaging floods that befell the city prior to 1849.

There are two principal types of swamps in the New Orleans area, inland swamps and mangrove swamps. Inland swamps typically occupy poorly drained areas enclosed by higher ground; either natural levee ridges (like Metairie Ridge) or, much older (Pleistocene age) Prairie Terraces. These basins receive fresh water from overflow of adjacent channels during late spring and early summer runoff. The trees growing in inland swamps are very sensitive to increases in salinity, even for short-lived periods. Continued subsidence allows eventual encroachment of saline water, gradually transforming the swamp to a grassy marsh. The relative age of the tree die-off is readily seen in the form of countless dead tree trunks, followed by stumps, which become buried in the marsh that supersedes the swamp. As a consequence, a thin veneer of marsh deposits often overlies extensive sequences of woody swamp deposits. The converse is true in areas experiencing high levels of sedimentation, such as those along the historic Mississippi and Atchafalaya River channels, where old brackish water marshes are buried by more recent fresh water swamp deposits. Swamp deposits typically contain logs, stumps, and arboreal root systems, which are highly permeable and conductive to seepage.

Mangrove swamps are the variety that thrives in salt water, with the two principal varieties being black and honey mangrove. Mangrove swamps are found along the distal islands of the Mississippi Delta, such as Timbalier, Freemason North, and the Chandeleur Islands, well offshore. Mangrove swamps also fringe the St. Bernard Marsh, Breton and Chandeleur Sounds, often rooting themselves on submerged natural levees. Mangrove swamps can reach heights of 20 to 25 feet in Plaquemines Parish. A typical soil column in a mangrove swamp consists of a thin layer of soft black organic silty clay with interlocking root zone that averages 5 to 12 inches thick. Tube-like roots usually extend a few inches above the ground surface. Thicknesses of five feet or more are common. Where they grow on sandy barrier beaches, the mangrove swamps thrive on the leeward side, where silts and clays intermingle with wash-over sands off the windward side, usually mixed with shells.

Surficial swamp deposits provide the least favorable foundations for structures and man-made improvements, like streets and buried utilities. Kolb and Saucier (1982) noted that the amount of structural damage in New Orleans was almost directly proportional to the thickness of surficial organic deposits (swamps and marshes). This peaty surface layer reaches thicknesses of up to 16 ft, as shown in Figure 3.12. Most of this foundation distress is attributable to differential settlement engendered by recent de-watering (discussed in Section 3.7.4).

3.1.5.3 Lacustrine Deposits

Lacustrine deposits are also deposited in a paludal environment of deltaic plains. This sequence most often occurs as marshes deteriorate (from lack of sediment) or subside (or both). These lakes vary in size, from a few feet in diameter to the largest, Lake Salvador (a few miles southwest of New Orleans), which measures 6 by 13 miles. Lake Pontchartrain (25

x 40 miles) is much larger, but is not a true marshland lake. The depths of these lakes vary from as little as 1.5 feet to about 8 feet (Lake Pontchartrain and Lake Borgne average 15 and 10 feet deep, respectively).

Small inland lakes within the marsh environment usually evolve from subsidence and erosion from wind shear and hurricane tides. Waves set up a winnowing action which concentrates the coarser material into the deepest portion of the lake. These lakes are generally quite shallow, often only a foot or two deep, even though up to a mile long. They are simply water-filled depressions on the underlying marsh, often identified in sampling by fine grained oozes overlying peats and organic clays of the marsh that preceded the transition to lake. The ooze become increasingly cohesive with age and depth, but is generally restricted to only 1 to 3 feet in thickness in small inland lakes.

Transitional lakes are those that become larger and more numerous closer to the actively retreating shoreline of the delta. These lake waters are free to move with the tides and currents affecting the open water of adjacent bays and sounds. Fines are often winnowed from the beds of these lakes and moved seaward, leaving behind silts and fine sands. Sediments in these lakes are transitional between inland lakes and the largely inorganic silty and sandy materials flooring bays and sounds.

Large inland lakes are the only lacustrine bodies where significant volumes of sediment are deposited. Principal examples would be the western side of Lake Borgne, Lake Pontchartrain, and Lake Maurepas, among others. Lacustrine clays form a significant portion of the upper 20 to 30 feet of the deltaic plain surrounding New Orleans. Lake Pontchartrain appears to have been a marine water body prior to the deposition of the Metairie Ridge distributary channel, which formed its southern shoreline, sealing it off from the Gulf. The central and western floor of Lake Pontchartrain is covered by clays, but the northern, eastern and southern shores are covered by silts and sands, likely due to the choppy wave-agitated floor of the shallow lake. Deeper in the sediment sequence oyster shells are encountered, testifying that saline conditions once existed when the lake was open to the ocean. The dominant type of mollusk within Lake Pontchartrain today is the clam *Rangia cuneata*, which favors brackish water. Dredging for shells was common in Lake Pontchartrain until the late 1970's.

During Hurricanes Katrina and Rita in 2005, wind shear removed extensive tracts of marsh cover, creating 118 square miles of new water surface in the delta. Forty-one square miles of shear-expanded pools were added to the Breton Sound Basin within Plaquemines Parish. This was more erosion and land loss than had occurred during the previous 50 years combined (Map USGS-NWRC 2006-11-0049).

3.1.6 Recognition Keys for Depositional Environments

Marsh deposits are typified by fibrous peats; from three principal environments: 1) fresh water marshes; 2) floating marsh – roots and grass sitting on an ooze of fresh water; and 3) saltwater marshes along the coast. The New Orleans marsh tends to be grassy marsh on a flat area that is “building down,” underlain by soft organic clays. Low strength smectite clays

tend to flocculate during brackish water intrusions, most commonly triggered by hurricanes making landfall in the proximate area.

Typical recognition keys for depositional environments have been summarized as follows.

- Cypress wood = fresh water swamp
- Fibrous peaty materials = marshes
- Fat Clays with organics; usually lacustrine. A pure fat clay has high water content (w/c) and consistency of peanut butter
- Interdistributary clays; paludal environments; lakes - Silt lenses when water is shallow and influenced by wind swept waves
- Lean clays CL Liquid Limit (LL) <50, silty and w/c <60%
- Fat clays CH Liquid Limit (LL) >50, no silt and w/c >70%

Abandoned meanders result in complex mixtures of channel sands, fat clay, lean clay, fibrous peat, and cypress swamp materials, which can be nearly impossible to correlate linearly between boreholes. The New Orleans District of the Corps of Engineers has historically employed 3-inch diameter Steel Shelby tubes and 5-inch diameter piston sampler, referring to samples recovered from the 5-inch sampler as their “undisturbed samples.” These are useful for characterizing the depositional environment of the soils. The larger diameter “undisturbed” samples are usually identified on boring logs and cross sections in the New Orleans District Design Memoranda by the modifier “U” for “undisturbed” samples (e.g. Boring prefixes X-U, UMP-X, MUE-X, MUG-X, and MUW-X).

3.1.7 Holocene Geology of New Orleans

The surficial geology of the New Orleans area is shown in Figure 3.13. The Mississippi River levees form the high ground, underlain by sands (shown as bright yellow in Figure 3.13). The old cypress swamps (shown in green) and grassy marshlands (shown in brown) occupied the low lying areas. The Mid-town area between the Mississippi and Metairie Ridge was an enclosed depression (shown in green) known as a “levee flank depression” (Russell, 1967). The much older Pleistocene age Prairie formation (shown in ochre) lies north of Lake Pontchartrain. This unit dips down beneath the city and is generally encountered at depths greater than 40 feet between the city (described previously).

The levee backslope and former swamplands north of Metairie Ridge are underlain by four principal stratigraphic units, shown in Figure 3.14. The surface is covered by a thin veneer of recent fill, generally a few inches to several feet thick, depending on location. This is underlain by peaty swamp and marsh deposits, which are highly organic and susceptible to consolidation. Entire cypress trunks are commonly encountered in exploratory borings, as shown in Figure 3.15. This unit contains two levels of old cypress swamps, discussed previously, and varies between 10 and 40 feet thick, depending on location. The clayey material beneath this is comprised of interdistributary materials deposited in a paludal (quiet water) environment, dominated by clay, but with frequent clay stringers. This unit pinches out in vicinity of the London Avenue Canal and increases in thickness to about 15 feet beneath the 17th Street Canal, three miles west. Occasional discontinuous lenses of pure clay are often encountered which formed through flocculation of the clay platelets when the swamp was inundated by salt water during severe hurricanes.

The area east of the Inner Harbor Navigation Canal (IHNC) is quite different (Figure 3.14), in that these deposits are dominated by fine-grained lacustrine deposits deposited in proto Lake Pontchartrain, and the Pine Island Sands are missing. These lacustrine materials extend eastward and are characterized by clays and silty clays with intermittent silt lenses and organics.

The lacustrine facies is underlain by the distinctive Pine Island Beach Sand, described previously. These relict beach sands thicken towards the east, closer to its depositional source. They reach a maximum thickness of about 30 ft. It thins westward towards Jefferson Parish, where it is only about 10 feet thick beneath the 17th Street Canal, as shown in Figure 3.14. The Pine Island sands are easily identified by the presence of mica in the quartz sand, and were likely transported from the mouth of the Pearl River by longshore drift (Saucier, 1963). Broken shells are common throughout the entire layer.

A bay sound deposit consisting of fine lacustrine clays begins just east of the Inner Harbor Navigation Canal; it begins near the 40 foot depth, has about a 10 foot thickness and continues to the west across the city, thickening along the way (Figure 3.14). It reaches its greatest thickness of about 35 feet just east of the 17th Street Canal. It is interesting to note that this area has experienced the greatest recorded settlement in the city, which may be attributable to dewatering of the units above this compressible lacustrine clay, increasing the effective stress acting on these materials (areas to the east are underlain by much more sand, which is less compressible).

The Holocene age deposits reach their greatest thickness just east of the 17th Street Drainage Canal where they are 80 feet thick (Figure 3.5). Undifferentiated Pleistocene deposits lie below these younger deposits.

For the most part, this area sits below sea level with the exception of the areas along old channels and natural levees. The Metairie-Gentilly Ridge lies above the adjacent portions of the city because it was an old distributary channel of the Mississippi River (Figure 3.1-upper). The same is true for the French Quarter and Downtown New Orleans, which are built on the natural sand levee of the Mississippi River.

Geology from the Inner Harbor Navigation Canal to the east becomes exceedingly complex. Although the surficial 10 feet consist of materials from an old cypress swamp, this is an area dominated by the Mississippi River and its distributaries, especially the old St. Bernard delta (See Figure 3.1-lower). Distributaries are common throughout the area and consist of sandy channels flanked by natural levees. 10-15 feet of interdistributary materials, mainly fine organic materials, are present between distributaries. Relic beaches varying in thickness from 10 to 15 feet are present below the interdistributary deposits. These beaches rest atop a 5-10 foot thick layer of nearshore deposits which are then followed by a thick sequence of prodelta clays leading out into the Gulf of Mexico.

3.1.8 Faulting and Seismic Conditions

Subsidence of the Gulf Geosyncline has led to numerous “growth” faults in South Louisiana. One group, the Baton Rouge Fault Zone (shown in Figure 3.7), is currently active

and passes in an east-west direction along the north shore of Lake Pontchartrain. Localized faulting is also common near salt domes. There has been no known faulting in the New Orleans area which has been active in Holocene times. The area is seismically quiescent. The earthquake acceleration with a 10% chance of being exceeded once in 250 years is about 0.04g.

3.2 Geologic Conditions at 17th Street Canal Breach

3.2.1 Introduction

The 17th St. Canal levee (floodwall) breach is one of New Orleans' more interesting levee failures. It is one of several levees that did not experience overtopping. Instead, it translated laterally approximately 50 feet atop weak foundation materials consisting of organic-rich marsh and swamp deposits. Trees, fences, and other features on or near the levee moved horizontally but experienced very little rotation, indicating the failure was almost purely translational in nature.

3.2.2 Interpretation of Geology from Auger Borings

A series of continuously sampled borings was conducted and logged using 3-inch Shelby tubes in the vicinity of the 17th St. Outlet Canal levee failure on 2-1-2006 (east side) and 2-7-2006 (west side) to characterize the geology of the materials serving as a foundation for the levee embankments and floodwalls. Drilling on the east bank took place just behind (east) of an intact portion of the levee embankment that had translated nearly 50 feet while drilling on the west side took place directly across the canal from the middle of the eastern breach. This drilling uncovered a wide range of materials below the embankments and provided insights into the failure.

Drilling on the east side of the levee was started at approximately 2-3 feet above sea level. A thin layer of crushed rock fill placed by contractors working for the U.S. Army Corps of Engineers to provide a working surface at the break site was augered through before reaching the native materials. Upon drilling at the east side of the levee, organic matter was encountered almost immediately and a fetid swamp gas odor was noted. This organic matter consisted of low-density peat, humus, and wood fragments intermixed with fine sand, silt, and clay, possibly due to wind shear and wave action from prehistoric hurricanes. This area appears to have been near the distal margins of a historic slough, as shown in Figure 3.16. At 4-6 feet, highly permeable marsh deposits were encountered and drilling fluid began flowing from a CPT hole several feet away, indicative of almost instantaneous conductivity at this depth. The CPT was sealed with bentonite before proceeding to prevent further fluid loss. The bottom of this sample was recovered as a solid 3-inch core of orange-red cypress wood indicating that this boring had passed through a trunk of stump of a former, but geologically young, tree.

A suspected slide plane was discovered at a depth between 8.3 and 11 feet below the ground surface depending on the location of the borings, indicative of an undulating slip surface. Gray plastic clays appeared to have been mixed with dark organics by shearing and

this zone was extremely mushy and almost soupy in texture. The water content was very high, on the order of 278%.

Organic rich deposits continue to a depth of about 20 feet below the surface while showing an increasing clay and silt content. Most clays are highly plastic with a high water content although there are lenses of lower plasticity clay, silt, and some sand. The variability of grain sizes and other materials is likely due to materials churned up by prehistoric storms. The clays are usually gray in color but vary and are olive, brown, dark gray, and black depending on the type and amount of organic content. Some organic matter towards the base of this deposit was likely roots that grew down through the pre-existing clays and silts or tree debris and that were mixed by prior hurricanes. Some woody debris came up relatively free of clays and closely resembled cypress mulch sold commercially for landscaping purposes. Full recoveries of material in this zone were rarely achieved in this organic rich zone. It appears that the low-density nature (less than water) of these soils caused them to compress due to sampling disturbances.

Most material below 21 feet was gray plastic clay varying from soft to firm and nearly pure lacustrine in origin. This clay included many silt lenses which tended to be stiffer and had some organics at 26 feet. It is likely that the silt and organics were washed into an otherwise quiet prehistoric Lake Pontchartrain by storms.

Sand and broken shells showed up at 30 feet in depth and continued to increase in quantity and size until 35.5 feet when the material became dirty sand with very little cohesion. This hole was terminated at 36 feet. These sands appear to be the Pine Island Beach Trend deposits, described in Section 3.1.3.

The geologic conditions beneath the 17th Street Canal breach are shown in Figures 3.17 thru 3.20. Figure 3.17 shows the relative positions of the cross sections presented in Figures 3.18 and 3.19. Figure 3.18 is a geologic section through the 17th Street Canal breach, extending into the canal. It was constructed using Brunton Compass and tape techniques commonly employed in engineering geology (Compton, 1962). In this section the landside of the eastern levee embankment translated laterally about 48 feet. The levee had two identifiable fill horizons, separated by a thin layer of shells, likely used to pave the old levee crest or the road next to the levee prior to 1915 (similar to the conditions depicted in Figure 4.18). A distinctive basal rupture surface was encountered in all the exploratory borings, as depicted in Figures 3.18 and 3.19. This rupture surface was characterized by the abrupt truncation of organic materials, including cypress branches up to two inches in diameter (shown in the inset of Figure 3.18). The rupture surface was between $\frac{3}{4}$ and 1 inch thick, and generally exhibited a very high water content (measured as 279% in samples recovered and tested). This material had a liquid consistency with zero appreciable shear strength. It could only be sampled within more competent materials in the Shelby Tubes. A brecciated zone three to four inches thick was observed in samples immediately above the rupture surface. This contained chunks of clay with contrasting color to the matrix materials, and up to several inches across, along with severed organic materials.

The geologic cross section portrayed in Figure 3.19 was taken on the north side of the same lot, using the same Brunton Compass and tape technique. It was located between 80 and 100 feet north of the previous section described above, as shown in Figure 3.17. In this

location the landside of the levee embankment translated about 52 feet laterally, to the east. These offsets were based on tape measurement made from the chain link right-of-way fences along the levee crest. No less than four distinct thrust planes were identified in the field, suggesting a planar, translational failure mode, as sketched in the cross section. As with the previous section, the old swamp deposits are noticeably compressed beneath the levee embankment, likely due to fill surcharge and the fact that the drainage canals have never been drained over their lifetime (in this case, since 1858 or thereabouts, described in Section 4.6). This local differential settlement causes the contact between the swamp deposits and the underlying lacustrine clays to dip northerly, towards the sheetpile tips supporting the concrete I-walls constructed in 1993-94. There was ample physical evidence that extremely high pore pressures likely developed during failure and translation of the levee block, in the form of extruded bivalve shells littering the ground surface at the second toe thrust, as shown in Figure 3.20 and indicated on the cross section (Figure 3.19).

Planar translational failures are typical of situations where shear translation occurs along discrete and semi-continuous low strength horizons (Cruden and Varnes, 1996). Additional evidence of translation is the relatively intact and un-dilated nature of the landside of the failed levee embankment, upon which the old chain link right-of-fence was preserved, as well as a substantial portion of the access road which ran along the levee crest, next to the concrete I-wall. Wherever we observed the displaced concrete I-wall in this area it was solidly attached to the Hoesch 12 steel sheetpiles, each segment of which was about 23 inches wide (as measured along the wall alignment) and 11 inches deep, with an open Z-pattern. The thickness of the sheets were about 7/16ths of an inch. The observed sheetpiles interlocks were all attached to one another. The entire wall system was quite stiff and fell backward (towards the canal) *after* translating approximately the same distance as the landslide of the levee embankment. The sheetpiles and attached I-walls formed a stiff rigid element. The sheetpiles were 23 ft-6 inches long and were embedded approximately 2 to 3 feet into the footings of concrete I-walls.

The geology of the opposite (west) bank was relatively similar except that the organics persist in large quantities, to a depth of 36 feet. The marsh deposits appeared deeper here and root tracks filled with soft secondary interstitial clay persisted to a depth of 39 feet. Sand and shells were first encountered at 40 feet and cohesionless sand was found at 41 feet. This hole was terminated at 42 feet.

3.2.3 Interpretation of Data from CPT Soundings

Six distinctive geologic formations are identified studying the Cone Penetrometer Test (CPT) soundings which were done in the vicinity of 17th Street Canal: Fill, swamp/marsh deposits, Intermixing deposits, lacustrine deposits, Pine Island beach sand deposits and Bay Sound deposits. The description and coverage of these geologic formations from CPT soundings are explained in the following paragraphs. These unit assignments are shown graphically in Figure 3.21.

FILL: Fill is not present in all CPT soundings. It is characterized by *stiff silty clay to sandy clay and sandy silt with some silt lenses*. It is differentiated from the swamp deposits by having little or no organic matter in its content. Along the breached area, the fill appears to be missing in the CPT soundings. Fill thickness is around 10 ft (down to -8 ft below sea level) on

the west bank of the 17th street Canal. Just north of the breached area (east bank), the thickness of the fill ranges from 14 ft to 16 ft (down to -10 ft). Fill materials for the drainage canals appear to have been placed in three sequences: 1) during the original excavation of the various canals, between 1833-1878; 2) after the 1915 Grand Isle Hurricane; and 3) after the October 1947 hurricane (the history of the drainage canals is described in Chapter 4, Section 4.6).

SWAMP/MARSH DEPOSITS: Marsh deposits consists of *soft clays, organic clays usually associated with organic material (wood and roots)*. The organic materials are readily identifiable by observing the big jumps in the friction ratios of the CPT's. The thickness of swamp/marsh deposits is around 9.5 ft on the west bank of the canal and 4 to 6 ft on the east bank of the canal. The depth at which swamp/marsh deposits encountered on banks ranges from approximately -8.5' (on the west side) to -10' (on the east side), using the NAVDD882004.65 datum.

INTERMIXING ZONE: This zone consists of mixture of *soft clays, silt lenses with little or no organic material*. The thickness of intermixing zone ranges from 3 ft to 8.5 ft on the east bank of the canal. No intermixing zone is interpreted on the west bank of the canal. However the contact between marsh and intermixing zone is highly irregular and should be correlated with borehole data.

LACUSTRINE DEPOSITS: Lacustrine deposits consist of *clays to organic clays with thin silt and fine sand lenses. No organic matter* is found in these deposits. The thickness of lacustrine deposits is around 17-19 ft on the west bank of the canal and 15-22 ft on the east bank of the canal. The depth at which lacustrine deposits encountered ranges from -17 (on the west side) to 14-23 (on the east side).

PINE ISLAND BEACH TREND SANDS: Beach sand is identified by its *sand and silty sand* content. It is easily recognized in the CPTs by a large jump in the tip resistance and a drop in the pore pressure. The depth at which beach sand encountered ranges from -37 (on the west side) to -36 ft (on the east side) and it has fairly uniform 6 ft of thickness.

BAY SOUND: This deposit contains *stiff organic clays and stiff clays*. It is easily recognized in the CPTs by a large drop in the tip resistance and an increase in the pore pressure. Bay sound deposits are only encountered on the east side of the canal and only top of bay sound deposits encountered in this area –not bottom. The depth at which these deposits encountered is around -42 ft (which appears to be uniform in this area).

3.3 Geologic Conditions at London Avenue Canal (North) Breach

3.3.1 Introduction

The London Ave. Outlet Canal Levee system catastrophically failed on its western bank just south of Robert E. Lee Blvd. during Hurricane Katrina between 9 and 10 AM on August 29, 2005. The hurricane induced a storm surge from the Gulf of Mexico that moved into Lake Pontchartrain and subsequently backed up into the canal. The levee failed at one location by translating laterally atop poor foundation materials, not by overtopping. The

break formed on the west bank levee just south of Robert E. Lee Blvd. The toe of this break appears to have thrust over the surrounding landscape 6-8 feet in places.

The east bank levee directly opposite this break translated by about two feet, but did not breach catastrophically. Here again, the movement was due to lateral translation, reflecting instability within the foundation soils. An imminent failure was likely but hydrostatic pressure was relieved by the break opposite this bank and a break on the east bank further south near Mirabeau Ave. Floodwall panels here have been displaced, tilted, and distressed.

Cohesionless beach sands from the micaceous Pine Island beach strand comprise the majority of the deposits beneath the London Ave. Canal Levee. These sands were quickly eroded and deposited in great quantities in the neighborhoods surrounding the breaks. Much of the sand was also likely in the bottom of the canal prior to the breaks.

3.3.2 Geology Beneath the Levees

A series of continuously sampled borings was conducted and logged using 3-inch Shelby tubes where cohesive soil was present. Cohesionless sands were sampled using the material recovered during the Standard Penetration Tests (SPT). CPTs were conducted alongside many of the other borings.

The first two feet of material appeared to be topsoil heavily influenced by modern vegetative growth. The material was a dark brown silty clay with many roots and organics and a relatively low water content.

The next 0.65 feet contained highly plastic and water-rich organic clay and contained what appeared to be the slide plane at 2.65 feet in depth. Although the slip surface was likely deeper under the levee, it was thrusting to the surface at this point. There was a return to the dark brown organic silty clay at this point, which continued to 3.1 feet where there was a strong contact. A gray clayey sand remained in the last 0.5 feet of the tube.

From 4-6 feet appeared to be a deposit of shallow marsh materials transitioning to beach sands from Lake Pontchartrain. The first part of the tube contained gray organic rich clays and silts with a fetid odor and transitioned to a relatively clayey gray sand. Cohesion dropped beyond 6 feet in depth and sampling was no longer possible using a Shelby tube. Sampling continued using an SPT split spoon sampler down to 44 feet where clays were again encountered. The entire layer of sand appeared to be beach sand. Shells were included throughout the layer and most sand was mica rich, likely brought in by long shore drift from the Pearl River. Shells were included throughout the layer and most sand was mica rich, likely transported by longshore drift from the Pearl River. This is the "relic beach" of the Pine Island Beach Trend described in Section 3.1.3.

The clay recovered from 44-46 feet was, silty, blue-gray in color, and very plastic. Sand and shell fragments were mixed in with this clay, possibly due to wave action and mixing due to storms. Additional boring logs show a lacustrine bay sound material at this depth. No sampling was conducted by our team below this depth. All recovered sediment was Holocene in age.

Boring logs from Design Manual 19A (U.S. Army Corps of Engineers, 1984) show similar results. In addition, the transitional layer of clayey sand is shown beneath the breach but not below adjacent unfailed sections of the levee. The marsh deposits and transitional zone extend up to 10 feet deeper beneath the breached levee (west) and distressed levee (east) than below the unbroken portions of the levee. Marsh deposits begin near the surface and transition to sand at around 10-15 feet in depth. The sand continues to around 45 feet where lacustrine bay sound material is found. This continues down to Pleistocene materials at 65-75 feet.

3.4 Geologic Conditions at London Avenue (South) Canal Breach

3.4.1 Introduction

The London Ave. Outlet Canal levee system catastrophically failed on its eastern bank just north of Mirabeau Ave. during Hurricane Katrina between 7 and 8 AM on August 29, 2005. This failure appears to have been induced by concentrated zone of underseepage, because the failure was relatively deep, and did not extend over a long zone of the canal. Nor was there any physical evidence of overtopping. The seepage appears to have been driven by high water level in the canal, caused by the storm surge coming up the canal from its mouth along Lake Pontchartrain.

Post failure reconnaissance revealed that micaceous sands from the Pine Island Beach Strand were eroded from this breach and, possibly, from within the canal where they were deposited throughout the surrounding neighborhood.

3.4.2 Geology Beneath the Levees

The section of levee incorporated in the London South breach is founded upon geology similar to the northern London Ave. Canal failure. The levee was constructed upon approximately ten feet of organic-rich cypress swamp deposits. Borings by the Corps of Engineers indicate that the swamp deposits extended three to five feet deeper below the failure area than the areas immediately adjacent to the breach (north or south of it). Unlike the London Avenue northern breach, where there is a transition of clayey sand between the marsh deposits and the underlying Pine Island Trend sands, there is a more definite transition at this location. These differences in foundation conditions are indicated on the boring logs within Design Manual 19A (U.S. Army Corps of Engineers, 1984).

3.5 Geologic Conditions along the Inner Harbor Navigation Canal

3.5.1 Introduction

Levees surrounding the Inner Harbor Navigation Canal (IHNC) were overtopped and breached catastrophically during Hurricane Katrina. Some of New Orleans' worst devastation occurred at two large breaches on the east side of the IHNC in the Lower Ninth Ward. These breaches washed houses from their foundations, leaving many blocks of the neighborhood as little more than piles of used lumber, destroyed automobiles, and other debris.

3.5.2 Geology

The geology beneath the IHNC levees is far more complex and variable than that of the foundation materials at the London Ave. and 17th St. Canals. The foundation materials here tend to be fluvially dominated by past distributaries of the Mississippi River with the exception of the area near Lake Pontchartrain. Conditions near the lake more resemble those under the London Ave. Canal but with a slightly thicker marsh deposit. The buried beach deposit is present below the marsh and eventually transitions into prodelta clays.

As with most modern fluvial systems, the geology of this Holocene deposit is complex and varies widely in both vertical and horizontal extent. The area was once covered by the marshes and swamps once common to the area. Organic fat clays are dominant and contain peat and other organic materials. Some wood is present but not in the quantities found at the 17th St. Canal site, indicating that marshes were more pronounced at this location. These deposits vary in thickness between 10-20 feet, depending on the location.

Interdistributary materials consisting largely of fat clays dominate much of the IHNC geology below the marsh deposits. This layer, which also contains zones/lenses of lean clays and silt, is approximately 30-35 feet thick.

A complex estuarine deposit exists below the interdistributary layer and is comprised of a complex mix of clays, silts, sands, and broken shell material. This deposit is about 30 feet thick and is underlain by Pleistocene deposits (undifferentiated, but commonly a stiff clay). Cross sections from The New Orleans District's Design Manual 02 Supplement 8 (U.S. Army Corps of Engineers, 1968, 1969, 1971) do not always do a good job of differentiating this material, but much of the material appears to be sand mixed with clays and silts. These deposits lie at sufficient depth as to preclude their having any significant impact on levee stability.

Abandoned distributaries cut across the IHNC in some locations. Materials in the old channels are highly variable. Although basal units usually consist of sands, upper units are heterogeneous layers of silts, clays, sandy silts, and silty sands. Natural levee deposits are commonly found around these old channels.

3.6 Paleontology and Age Dating

3.6.1 Introduction

Micropaleontology was used in conjunction with carbon 14 dating to determine both the age and depositional environment of the sediments below levee failure sites in New Orleans, LA. Foraminifera, single-celled protists that secrete a mineralized test or shell, were identified as these organisms grow in brackish or marine settings but not freshwater. Their presence in sediments indicate that they were deposited in-situ or were transported from brackish Lake Pontchartrain or marine environments by Hurricanes. Palynology, the identification and study of organic-walled microfossils, commonly pollens and spores, was conducted to aid in the re-creation of paleoenvironments beneath the levees. Macrofossils of the phylum *Mollusca*, including classes *Gastropoda* and *Bivalvia* are common in sands of the Pine Island Trend (Rowett, 1958). Most recovered samples contained heavily damaged shells or fragments.

3.6.2 Palynology

Although varying sediment types including clays, peats, and sands were studied, similar palynomorphs were found throughout the samples. These samples came from different depths and locations throughout New Orleans. The commonalities between the sediments may be due to transportation of the palynomorphs by wind and water or the mixing of materials by hurricanes. Pollens of the family *Taxodiaceae*, genus *Cupressacites* (cypress) are common. Species of cypress are common in perennially wet areas such as swamps. Cypress is common throughout the swamps of the Gulf Coast Region. Cypress wood, including trunks, roots, and stumps, was unearthed by scour during the levee failures and subsequent construction to temporarily patch the levees. Samples recovered in 3" Shelby tubes commonly included cypress fragments resembling commercially available landscaping mulch and cores of intact wood. Cypress trees are freshwater and die if exposed to salt water for a prolonged amount of time.

Dinocysts/Dinoflagellates were also discovered among the samples taken for palynology. Dinoflagellates are single-celled algae belonging to the Kingdom Protista. They live almost exclusively in marine and brackish water environments, with very few freshwater species. The discovery of these organisms was not surprising, given the close proximity to brackish Lake Pontchartrain (essentially a bay). On the other hand, several exclusively marine species that live in the open ocean were recovered. These species were transported a far distance inland, indicating transport by a catastrophic event, possibly a hurricane storm surge or tsunami.

3.6.3 Foraminifera

Foraminifera were identified in the Pine Island Trend, a micaceous quartz beach sand that was deposited in the Holocene Gulf of Mexico by the Pearl River of Mississippi. This sand was subsequently formed into a large sand spit by long shore drift, separating Lake Pontchartrain from the rest of the Gulf of Mexico (Saucier, 1994). Lake Pontchartrain is a brackish body of water with only a small connection to the Gulf. Agglutinated, planispiral, and uniserial foraminifera were discovered where the sand grades into the silts and clays deposited in the low energy environments of Lake Pontchartrain. Although foraminifera are abundant at these locations, their diversity is low. This is indicative of a stressed environment and is not surprising, given the brackish nature of Lake Pontchartrain.

3.6.4 Carbon 14 age dating

We are awaiting the results of six C14 age dating by the NSF-funded age dating laboratory at the University of New Mexico in Albuquerque, NM. These are samples of the cypress wood and fibrous peats recovered at the 17th Street Canal failure area.

3.7 Mechanisms of Ground Settlement and Land Loss in Greater New Orleans

3.7.1 Settlement Measurements

URS Consultants (2006) in Baton Rouge recently completed a study for FEMA of the relative ground settlement in New Orleans since 1895, using the Brown (1895) map, which

has 1 foot contours and extends north to the Lake Pontchartrain shoreline. This comparison was made by creating Digital Elevation Models (DEMs) of the 1895 map (Figure 3.22) relative to Mean Gulf Level against the 1999/2002 DEM extracted from LiDAR data and New Orleans network of benchmarks. The resulting product was a map noting relative settlement (in feet) between 1895 and 1999, shown in Figure 3.23. This study suggests that the entire city has settled between 2 and 10 feet. During this same interim, sea level has risen approximately 12 inches. The area with the greatest settlement (> 8 feet) was north of I-610 in the Lakeview area and north of Mirabeau Ave. in the Gentilly area, exclusive of the 1931 fill along Lake Pontchartrain (which extends a half mile into the Lake).

3.7.2 Tectonic Subsidence

Tectonic subsidence is caused by sediment compaction at great depths (Figure 3.24). Salt and muds flow towards the continental shelf. Pressure ridges and fold belts develop; which are akin to sitting on a peanut butter and jelly sandwich and watching material ooze out and shift. The Continental Slope and Shelf is blanketed by large subaqueous landslides.

3.7.3 Lystric Growth Faults

As compacting materials move seaward, the ground surface drops. If sediment is not added at the ground surface, the seaward side of these features gradually subsides below sea level. The delta's lystric growth faults have been grouped into bands thought to be more or less related to one another. The relatively recent emergence of the Baton Rouge Fault Zone along the northern shore of Lake Pontchartrain, thence towards Baton Rouge, is the most striking example, and one of the furthest inland (Figures 3.25 and 3.26).

3.7.4 Compaction of Surficial Organic Swamp and Marsh Deposits

The interdistributary sediment package covering the old back swamps around New Orleans is highly compressible and the neighborhoods built on these materials exhibit obvious signs of differential settlement. This is particularly true of the West End, Lakeview, City Park, Fillmore, St. Anthony, Dillard, Milneburg, Pontchartrain Park, Desire, and Gentilly neighborhoods flanking Lake Pontchartrain. Most of this settlement is ascribable to oxidation-induced settlement of underlying peaty soils, caused by local drawdown of the ground water table, as sketched in Figure 3.27. The amount of post-development settlement is more-or-less proportional to the thickness of the peaty surface layer, shown in Figure 3.12. It varies in thickness from a few feet to as much as 20 feet, depending on location (WPA-LA, 1937; Kolb and Saucier, 1982).

The mechanisms promoting surficial settlement in lower New Orleans are thought to be: 1) drainage of the near surface soils, through simple near-surface dewatering and the storm water collection system; and 2) biochemical oxidation of organic materials above the [lowered] water table. Simple drainage of the surficial peaty soils can induced consolidation of up to 75% of their original thickness (Kolb and Saucier, 1982), which in of itself, could account for up to 12 ft of settlement, if the local water table was lowered >15 feet. But, biochemical oxidation continues afterwards, with greater severity during extended periods of drought, as occurred in the late 1990s-early 2000s around New Orleans. Oxidation continues until only the mineral constituents of the soil are left remaining.

Dense urban development also leads to increased subsidence because the absorptive capacity of the peaty soils is decreased by the mass implementation of impervious surfaces, such as streets, parking lots, sidewalks, roofs, driveways, etc. Increasing the area of impervious surfaces decreases overall seasonal infiltration and increases the peak runoff through hardened impervious surfaces. As a consequence, the Sewerage & Water Board of New Orleans had to continually increase the capacity of their drainage collection, conveyance and discharge system during the post-1945 period. These examples are from the Lakeview area adjacent to the 17th St. Canal failure, where the ground appears to have settled 10 to 16 inches since 1956.

The Lakeview and Gentilly neighborhoods were intensely developed in the post World War II era, mostly between 1946-70 (although infilling of newer structures continued up through 2005, as older structures were torn down). Most residential structures built in lower New Orleans after the mid-1950s are concrete slabs founded on wood pilings 6 to 8 inches in diameter, driven about 30 feet deep (Waters, 1984). From inspection, it appears that the ground beneath the foundations has settled 10 to 40 inches over the past 50 +/- years since these homes were constructed. This development was accompanied by a lowering of the ground water table to accommodate normal living conditions and combat mildew and mold in the crawl spaces beneath the homes (Figure 3.28 - upper). Since the historic groundwater table was at or within a few inches of the ground surface in this area, the lowering of the water table by 2 to 10 feet in this area hastened near-surface settlement through oxidation of the organic rich peat soils underlying the area.

As the peats oxidize, the ground settles, creating a depressed area beneath pile supported homes (Figure 3.28-upper). Groundwater pumping, drainage, and structural and earthen surcharges all contribute to the observed settlement. Historic measurements of ground settlement in the Kenner area of Jefferson Parish are shown in Figure 3.29.

During the 130 to 170 years since the drainage canals were constructed upon what became the Lakeview and Gentilly areas, these channels have never been drained for any significant period of time, because they were open to Lake Pontchartrain. As a consequence, the peaty soils immediately beneath these canals (17th Street, Orleans, and London Avenue) and Bayou St. John have not experienced significant near-surface settlements like those fomented by oxidation of peaty soils in the adjoining neighborhoods, although they have experienced gross ground settlement due to the other causes described in Section 3.7.

This history of near-continual ground settlement necessitated raising of the old drainage canal embankments on three occasions in the 20th Century, following hurricane-induced flooding from storm surges off Lake Pontchartrain, in: 1915, 1947, and 1965. Earth fill was placed upon the levee embankments in 1915 and 1947. After flooding associated with Hurricane Betsy in 1965 steel sheetpiles were used in selective zones to increase the freeboard for Category 3 storm surge (a figure that shifts each decade, as new information and models are developed). In the 1990s sheetpile-supported concrete I-walls were constructed along the crests of the drainage canals and on either side of the IHNC.

3.7.5 Structural Surcharging

An interesting aspect of the recent URS (2006) study for FEMA is the marked increase in settlement noted in the Central Business District, where tall structures are founded

on deep piles. This area settled 5 inches in 100 years, but much less further away from the city's tallest and heaviest structures. The sandy natural levees along the Mississippi River even settled 2 inches; likely due to surcharging by the Corps' Mississippi River & Tributaries Project (MR&T) sequences of levee enlargements, between 1928-60.

3.7.6 Extraction of Oil, Gas, and Water

Since the 1960s groundwater withdrawal has been recognized as contributing to subsidence of the Gulf Coast area, especially adjacent to deep withdrawal points for industrial consumption (Kazmann and Heath, 1968). More recently, R.A. Morton of the USGS has blamed oil and gas extraction for the subsidence of the Mississippi Delta. Morton has constructed convincing correlations between petroleum withdrawal and settlement rates on the southern fringes of the delta, near the mouth of the Mississippi River (Morton, Buster, and Krohn, 2002). But, other factors are likely involved as well, as petroleum withdrawal alone cannot account for marked settlement well inland of Lake Pontchartrain, where little withdrawal has occurred. Figure 3.30 presents Saucier's (1994) map of the Mississippi Delta, which summarizes the structural geologic framework of the area. This shows salt basins, salt domes, and active growth faults that pervade the delta region. Solutioning of salt diapirs and seaward migration of low density contrast materials likely exacerbate settlement, but more slowly than fluid/gas withdrawal.

3.7.7 Coastal Land Loss

The U.S. Geological Survey's National Wetlands Research Center (USGS-NWRC) has about 100 years of land loss information. Since 1973, satellites have allowed monitoring of sediment expulsion from the delta and the nefarious shoreline, which is continuously sinking. The USGS-NWRC has been monitoring coastal land loss over the past 50 years using 1956 and 1978 imagery published by Cahoon and Groat (1990) and LANDSAT Thematic Mapper satellite imagery from 1993 and 2000 (Barras et al., 2003).

Coastal lands loss is a high visibility problem along the Gulf Coast, especially in the Mississippi Delta.

- USGS and NGS state that the approximate rate of subsidence is between 1/3" to 1/2" per year; or about 4.2 ft/100 yrs
- Sea level rise is running about 1 ft/100 yrs (Burkett, Zilkowski, and Hart, 2003)
- 15% of New Orleans is already more than -10 ft below sea level (URS, 2006)
- The average current rate of coastal land loss is between 25 and 118 square miles per year (the record of 118 mi² being a result of Hurricanes Katrina and Rita in 2005)
- The 2050 Reclamation Plan would restore 25 to 30 mi² over the next 40 to 50 yrs at a cost of \$14 billion

The USGS National Wetlands Research Center has determined that Hurricane Katrina created as much new standing water area in the Mississippi Delta (below sea level) as occurred naturally over the previous 50 years! This was due to increased traction shear, which tore out large tracts of peat bogs, to depths of several feet (USGS-NWRC, 2006).

3.7.8 Negative Impact of Ground Settlement on Storm Surge

As large tracts of land along coastal Louisiana sink below sea level, less protection is afforded inland areas from the destructive impacts of storm surges caused by hurricanes. The absolute level of storm surges on the Louisiana Coast is also likely exacerbated by the loss of coastal vegetation, such as cypress swamps, which mollify wave energy through mechanical obstruction and tortuous flow path (increased boundary shear) as high water sweeps onto the land. The diminution of storm surge height would depend on the speed and duration of the storm as it makes landfall, and the density and height of the cypress swamps and the vegetation they support.

Many figures have been cited in the non-technical literature in regards to this “protective impact;” the most common being that every 4-1/2 miles of mature cypress swamp absorbs one foot of storm surge coming from the Gulf (Hallowell, 2005). Although the concept of storm surge mollification through turbulent boundary shear at the ground surface is conceptually possible, we were unable to find any measurements that quantified this effect through credible scientific study of historic storm events (NRC, 2006). Observations made during and after Hurricane Katrina, may, however, help to fill this data void.

3.7.9 Conclusions about Ground Settlement

Multiple physical factors have combined to cause marked historic settlement of the New Orleans area. These include:

- 1) The average silt load of the Mississippi River (550 million tons [mt] per year prior to 1950; now 220 mt/yr) causes continuous crustal loading of the Mississippi River Delta, causing isostasy-driven settlement, which has been recognized since 1937 (Meade and Parker, 1985; Russell, 1940, 1967).
- 2) Tectonic compaction caused by sediment compaction at great depths, with associated pressure ridges and fold belts.
- 3) Subsidence along the seaward side of lystric growth faults perturbing the Mississippi Delta.
- 4) Drainage of near-surface soils causing an increase in effective stress and resulting primary consolidation
- 5) Oxidation of near-surface peaty soils due to lowering of the groundwater table in developed areas, or drainage of historic marshes and swamp lands. This component is often exacerbated by New Orleans residents who routinely fill in portions of their yards adjacent to protruding foundations (Figure 3.28), driveways and sidewalks, creating additional loads on the compressible materials lying beneath them.
- 6) Consolidation of soft compressible soils (with high water contents), due to surcharging by earth filling and other man-made improvements.
- 7) Structural surcharging. Settlements measured in vicinity of downtown high rise structures suggests that a portion of the observed settlement may also emanate from deeper horizons, caused by loads transferred to those horizons along friction piles and caissons for heavy structures.

- 8) Fluid extraction of oil, gas, and water from the subsurface. Extraction of fluids and natural gas is a pressure depletion that increases effective stresses acting on underlying sediments, hastening consolidation.
- 9) Solutioning of salt diapirs (salt domes) and seaward migration of low density contrast materials (salt and mud), as well as large subaqueous slope movements on the continental slope and shelf. When large volumes of material move laterally, adjoining areas drop to compensate for the volumetric strain.

3.8 References

- American Institute of Professional Geologists. (1993). *A Citizen's Guide to Geologic Hazards*. Arvada, Colorado, 134 p.
- Barras, J., Beville, S., Britsch, D., Hartley, S., Hawes, S., Johnston, J., Kemp, P., Kinler, Q., Martucci, A., Porthouse, J., Reed, D., Roy, K., Sapkota, S., and Suyhada, J. (2003). *Historical and Projected Coastal Louisiana Land Changes: 1978-2050*, USGS Open File Report 03-334, U.S. Geological Survey National Wetlands Research Center, Baton Rouge LA.
- Brown, L.W. (1895). *Contour Map of New Orleans (Plate 11)*, Report of the Advisory Board on Drainage of the City of New Orleans, Louisiana, New Orleans Engineering Committee.
- Burkett, V.R., Zilkowski, D.B., and Hart, D.A. (2003). "Sea-Level Rise and Subsidence: Implications for Flooding in New Orleans, Louisiana." *USGS Subsidence Interest Group Conference, Proceedings of the Technical Meeting*, K. R. Prince and D.L. Galloway (Eds.), Galveston TX, 63-70.
- Cahoon, D.R., and Groat, C.G. (1990). *A Study of Marshy Management Practice in Coastal Louisiana*, Vol II, Technical Description, OCS study MMS 90-0076, Minerals Management Service, New Orleans LA.
- Coastal Environments, Inc. (2001). *Southeast Louisiana Fault Study*, Consultant's Report for U.S. Army Corps of Engineers, New Orleans District, Contract No. DACW-29-00-C-0034 (February).
- Coleman, J.M. (1988). "Dynamic Changes and Processes in the Mississippi River Delta." *Geological Society of America Bulletin*, 100(7), 999-1015.
- Coleman, J.M. and Galiano, S. M. (1964). "Cyclic Sedimentation in the Mississippi River Deltaic Plain." *Transactions of the Gulf Coast Association of Geological Societies*, 14, 67-80.
- Coleman, J.M. and Robert, H.H. (1991). "The Mississippi River Depositional System: A Model for the Gulf Coast Tertiary." *An Introduction to Central Gulf Coast Geology*, D. Goldthwaite (Ed.), New Orleans Geological Society, New Orleans LA, 99-121.
- Compton, R.R. (1962). *Manual of Field Geology*. John Wiley & Sons, New York.
- Cruden, D.M., and Varnes, D.J. (1996). *Landslide Types and Processes Special Report 247*. In A.K. Turner and R.L. Schuster, eds. *Landslides Investigation and Mitigation*. Transportation Research Board. National Academies Press.

- Draut, A.E., Kineke, G.C., Velasco, D.W., Allison, M.A., and Prime, R.J. (2005). "Influence of the Atchafalaya River in Recent Evolution of the Chenier Plain inner Continental Shelf, Northern Gulf of Mexico." *Continental Shelf Research* 25, 91-112.
- Dunbar, J.D., Blaes, M.R., Duit, S.E., May, J.R., and Stroud, K.W. (1994). *Geological Investigation of the Mississippi River Deltaic Plain*, Technical Report GL-84-15, U.S. Army Engineer Waterways Experiment, Vicksburg MS.
- Gould, H.R., and Morgan, J.P. (1962). "Coastal Louisiana Swamps and Marshes." *Geology of the Gulf Coasts and Central Texas and Guidebook of Excursions*, E. Rainwater, and R. Zingula (Eds.), Houston Geological Society, Houston TX.
- Gould, H.R. (1970). "The Mississippi Delta Complex" *Deltaic Sedimentation: Modern and Ancient, SEPM Special Publication 15*, J.P. Morgan (Ed.), Society of Economic Paleontologists and Mineralogists, Tulsa Oklahoma, 3-30.
- Hallowell, C. (2005). *Holding Back the Sea: the Struggle on the Gulf Coast to Save America*, Harper Perennial, New York.
- Interagency Performance Evaluation Task Force (IPET). (2006). *Performance Evaluation Status and Interim Results.* Report 2 of a Series, Performance Evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System., U.S. Army Corps of Engineers, Washington DC.
- Kazmann, R.G., and Heath, M.M. (1968). "Land Subsidence Related Ground-Water Offtake in the New Orleans Area." *Transactions of the Gulf Coast Association of Geological Societies*, 18, 108-113.
- Kolb, C.R. (1958). *Geology of the Mississippi River Deltaic Plain, Southeastern Louisiana*, Technical Report 3-483, Vol. 1, U.S. Army Engineer Waterways Experiment Station, Vicksburg MS.
- Kolb, C.R. and Shockley, W.G. (1959). "Engineering Geology of the Mississippi Valley." *ASCE Transactions*, 124, 633-656.
- Kolb, C.R., Smith, F.L., and Silva, R.C. (1975). *Pleistocene Sediments of the New Orleans-Lake Pontchartrain Area*, Technical Report No. S-75-6, U.S. Army Engineer Waterways Experiment Station, Vicksburg MS.
- Kolb, C.R., and Van Lopik, J.R. (1958). *Geology of the Mississippi River Deltaic Plain, Southern Louisiana*, Technical Report No. 3-483, U.S. Army Engineer Waterways Experiment Station, Vicksburg MS.
- Meade, R.H., and Parker, R.S. (1985). "Sediment in Rivers of the United States." *National Water Summary 1984 – Water Quality Issues*, U.S. Geological Survey, 49-60.
- Moore, N. R. (1972) *Improvement of the Lower Mississippi River and Tributaries 1931-1972*, Department of the Army Corps of Engineers, Mississippi River Commission, Vicksburg MS.
- National Research Council. (2006). *Drawing Louisiana's New Map: Addressing Land Loss in Coastal Louisiana*, National Academies Press, Washington DC.
- Russell, R.J. (1940). "Quaternary History of Louisiana." *Geological Society of America Bulletin*, 51, 1199-1234.

- Rowett, C.L. (1957). "A Quaternary Molluscan Assemblage from Orleans Parish, Louisiana." *Transactions of the Gulf Coast Association of Geological Sciences*, 7, 153-164.
- Russell, R. J. (1967). *River Plains and Sea Coasts*, Univ. of California Press, Los Angeles, CA.
- Saucier, R.T. (1963). *Recent Geomorphic History of the Pontchartrain Basin. Louisiana, Coastal Studies Series No 9*, Louisiana State University Press, Baton Rouge LA.
- Saucier, R.T. (1994). *Geomorphology and Quaternary Geologic History of the Lower Mississippi Valley*. U.S. Army Corps of Engineers Waterways Experiment Station, Vicksburg MS.
- Terzaghi, K. (1956). "Varieties of Submarine Slope Failures." *Proceedings of the Eighth Texas Conference on Soil Mechanics and Foundation Engineering*, 1-41.
- URS Consultants. (2006). *A Century of Subsidence: Change in New Orleans DEMs Relative to MGL 1895 to 1999/2002*. GIS Maps Prepared for The Federal Emergency Management Agency, URS Consultants, Baton Rouge LA.
- U.S. Army Corps of Engineers. (1968) *Lake Pontchartrain, LA and Vicinity Lake Pontchartrain Barrier Plan, Design Memorandum No 2, General Supplement No. 8, Inner Harbor Navigation Canal Remaining Levees*, New Orleans District, New Orleans LA.
- U.S. Army Corps of Engineers. (1969) *Lake Pontchartrain, LA and Vicinity Lake Pontchartrain Barrier Plan, Design Memorandum No 2, General Supplement No. 8, West Levee Vicinity France Road and Florida Avenue*, New Orleans District, New Orleans LA.
- U.S. Army Corps of Engineers. (1971) *Lake Pontchartrain, LA and Vicinity Lake Pontchartrain Barrier Plan, Design Memorandum No 2, General Supplement No. 8, Modification of Protective Alignment and Pertinent Design Information I.H.N.C. Remaining Levees West Levee Vicinity France Road and Florida Avenue Containerization Complex*, New Orleans District, New Orleans LA.
- U.S. Army Corps of Engineers. (1984). *General Design London Avenue Outfall Canal, Design Memorandum No. 19A*, New Orleans District, New Orleans LA.
- U.S. Army Corps of Engineers. (1990). *General Design 17th St. Outfall Canal (Metairie Relief), Design Memorandum No. 20*, New Orleans District, New Orleans, LA.
- U.S. Geological Survey (USGS)-National Wetland Research Center(NWRC). (2006). *Water Area Changes in Southeastern Louisiana after Hurricanes Katrina and Rita Detected with Landsat Thematic Mapper Satellite Imagery*, Map ID: USGS-NWRC 2006-11-0049.
- Waters, R. K. (1984). *Handbook for Pile Supported Foundations*, G.G. Printing Co., Kenner, Louisiana LA.
- Works Projects Administration of Louisiana. (1937). *Some Data in Regard to Foundations in New Orleans and Vicinity*, New Orleans LA. 243 p.

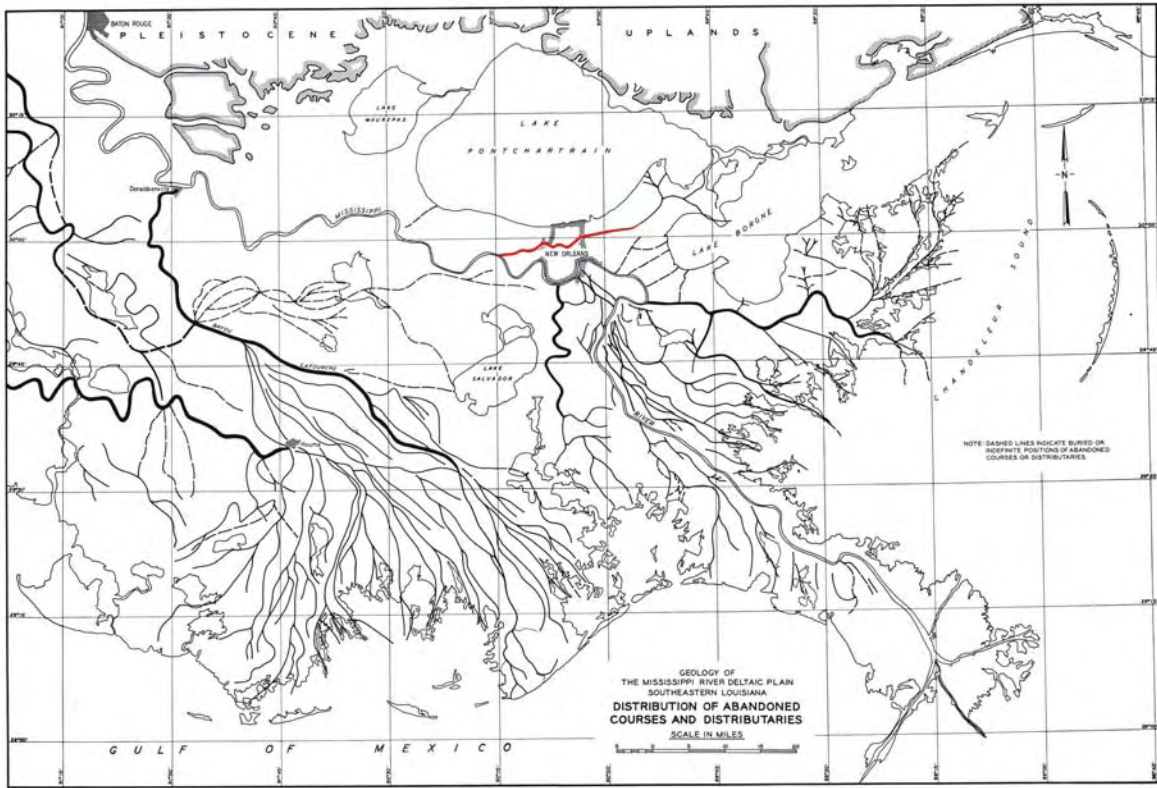


Figure 3.1 (upper): Areal distribution of abandoned channels and distributaries of the Mississippi River (from Kolb, 1958). The Metairie Ridge distributary channel (highlighted in red) lies between two different depositional provinces in the center of New Orleans (shown in Figure 3.6).

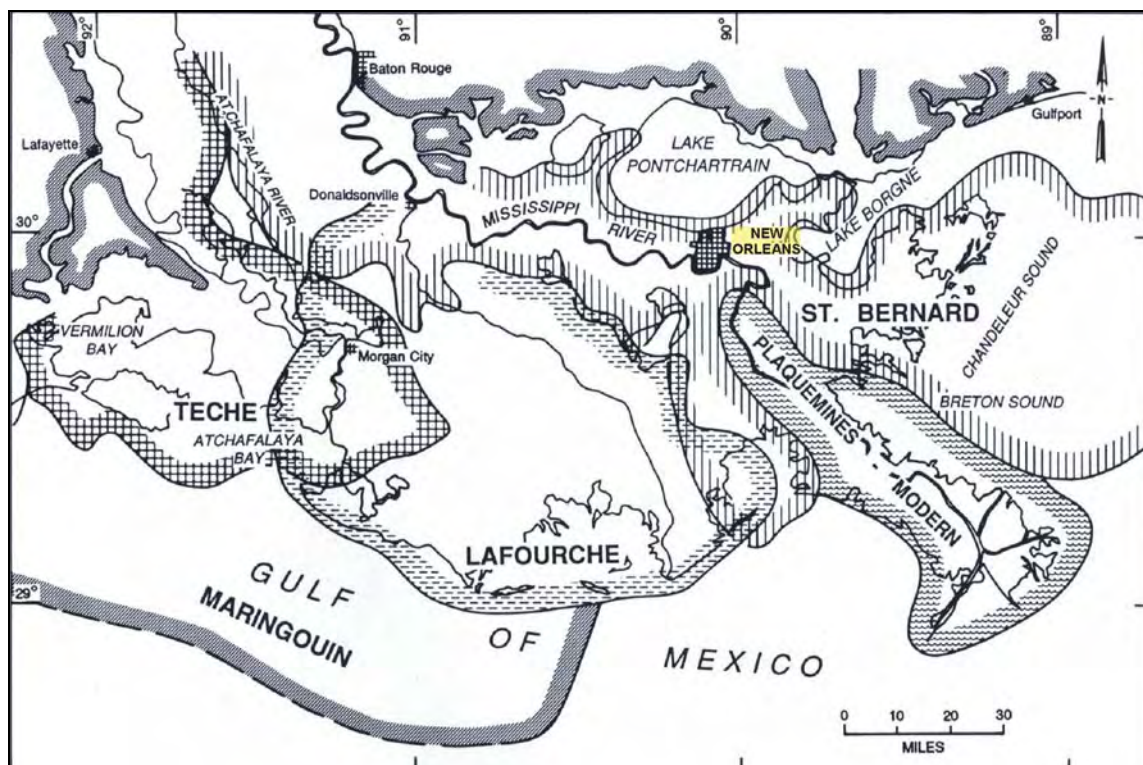


Figure 3.1 (lower): Major depositional lobes identified in lower Mississippi Delta around New Orleans, taken from Saucier (1994).

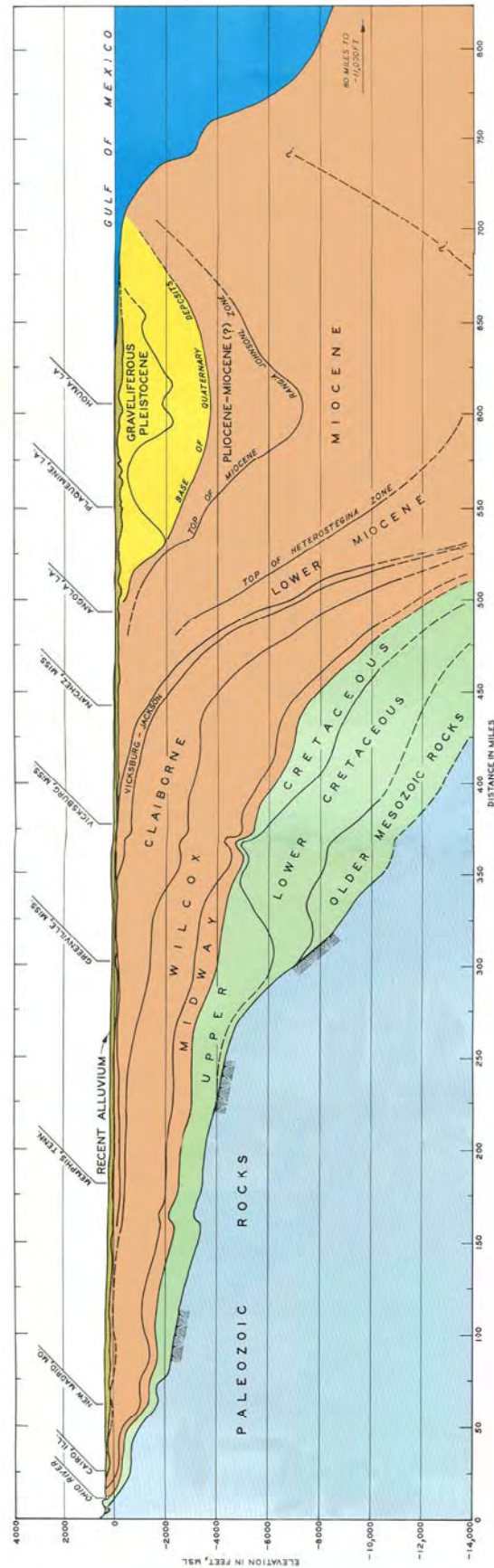


Figure 3.2: North-south geologic cross section through the central Gulf of Mexico Coastal Plain, along the Mississippi River Embayment (from Moore, 1972). Note the axis of the Gulf Coast Geosyncline beneath Houma, LA, southwest of New Orleans. In this area the Quaternary age deposits reach a thickness of 3600 ft.

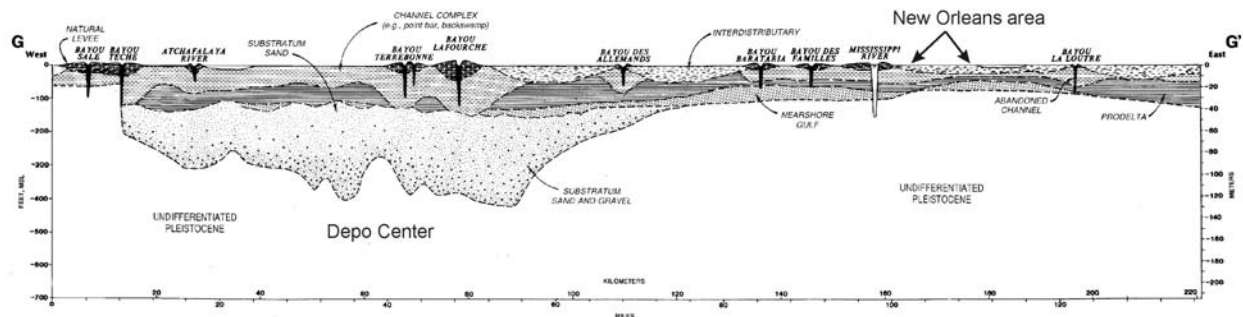


Figure 3.3: Transverse cross section in a west to east line, across the Mississippi River Delta a few miles south of New Orleans, cutting across the southern shore of Lake Borgne (modified from Saucier, 1994). New Orleans is located on a relatively thin deltaic plain towards the eastern side of the delta's depositional center, which underlies the Atchafalaya Basin, west of New Orleans.

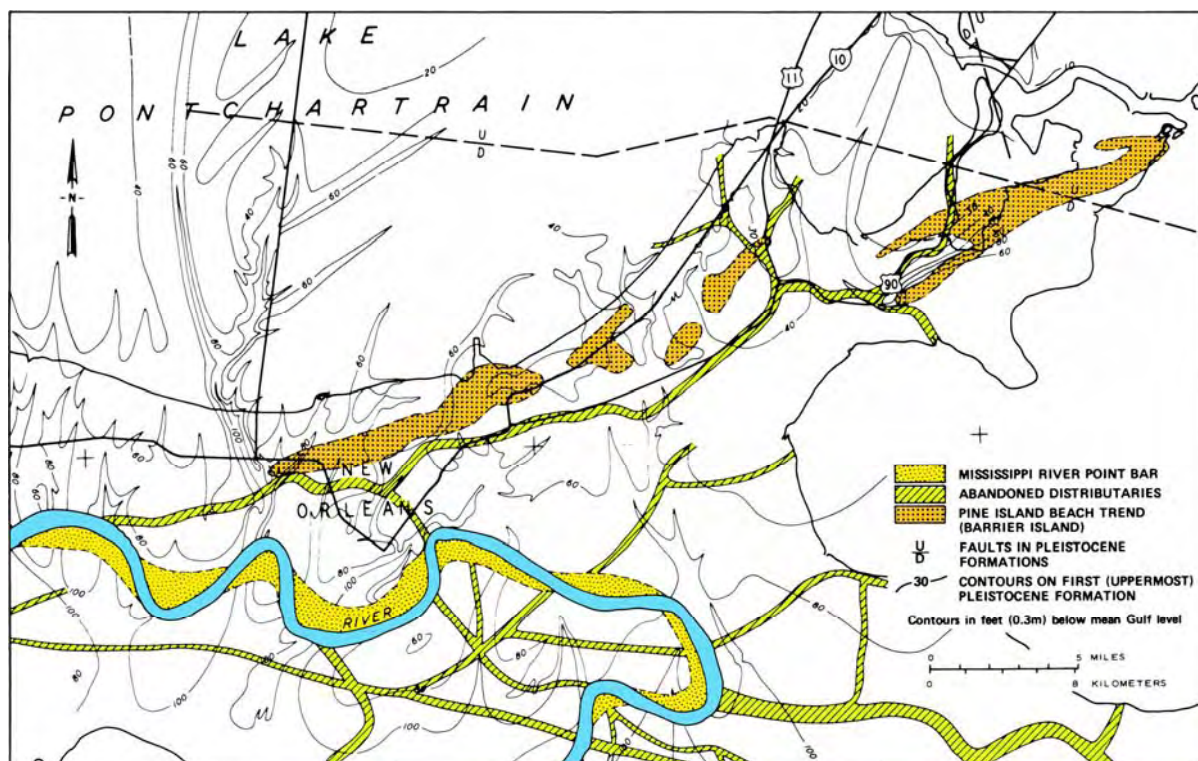


Figure 3.4: Pleistocene geologic map of the New Orleans area, taken from Kolb and Saucier (1982), modified from Kolb and Saucier (1982). The yellow stippled bands are the principal distributary channels of the lower Mississippi during the late Pleistocene, while the present channel is shown in light blue. The Pine Island Beach Trend is shown in the ochre dotted pattern. Depth contours on the upper Pleistocene age horizons are also shown.

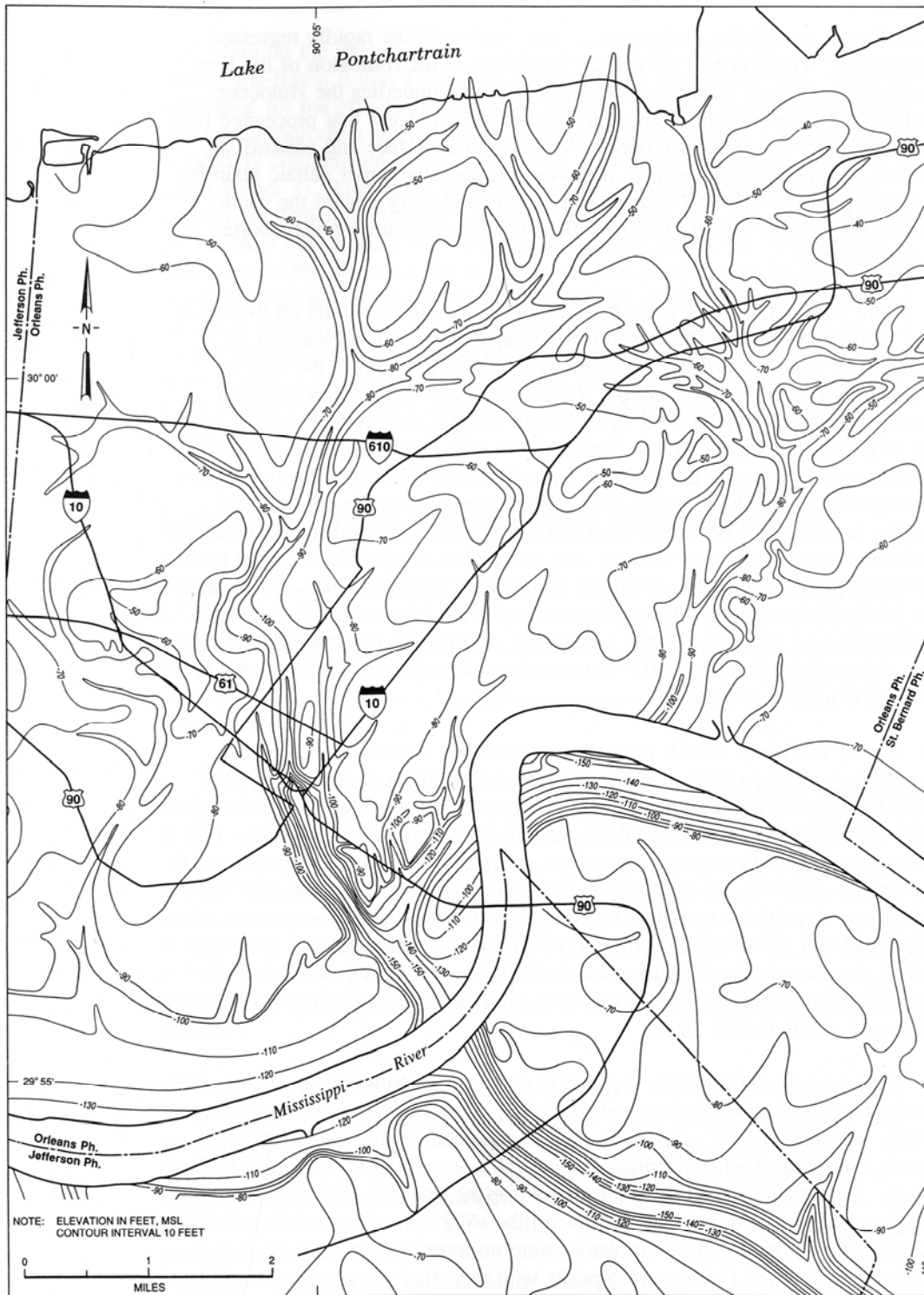


Figure 3.5: Contours of the entrenched surface of the Wisconsin glacial age deposits underlying New Orleans, taken from Saucier (1994). Note the well developed channel leading southward, towards what used to be the oceanic shoreline. This channel reaches a maximum depth of 150 feet below sea level.

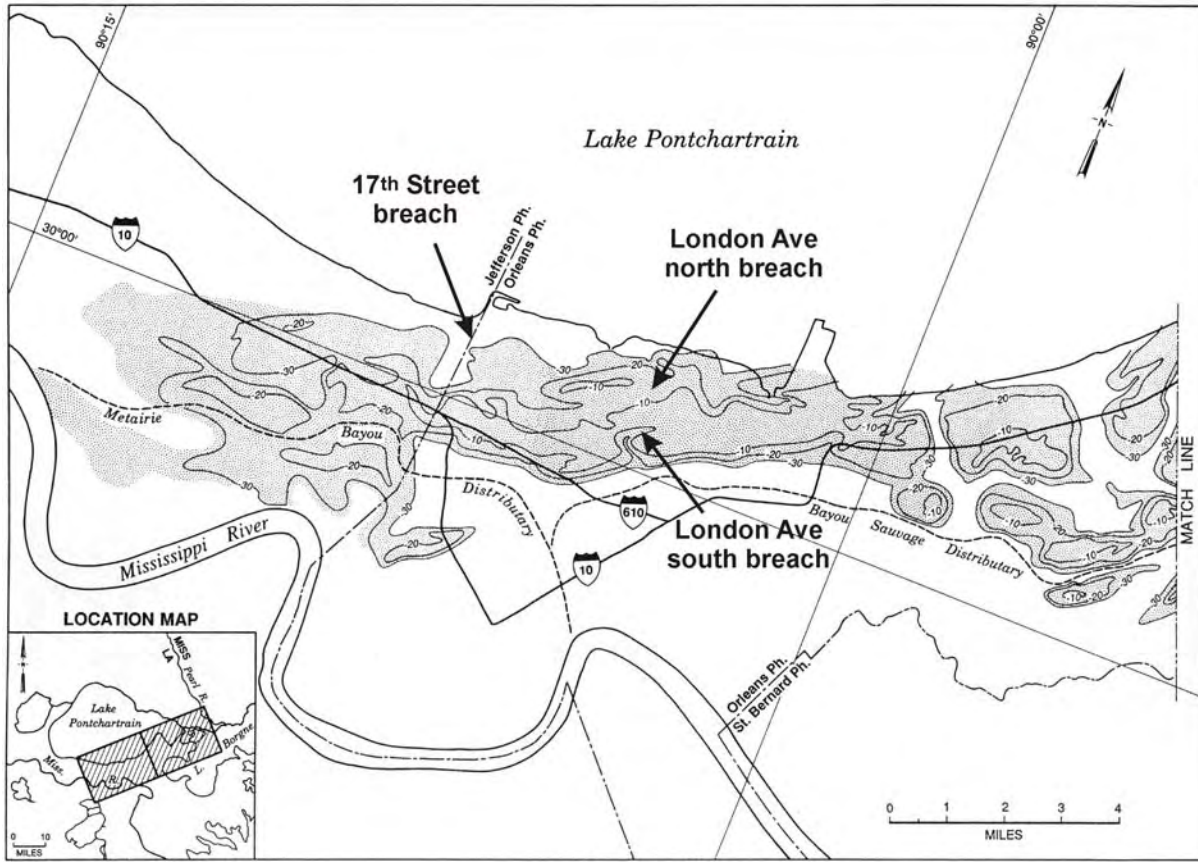


Figure 3.6: Areal distribution and depth to top of formation isopleths for the Pine Island Beach Trend beneath lower New Orleans, modified from Saucier (1994).

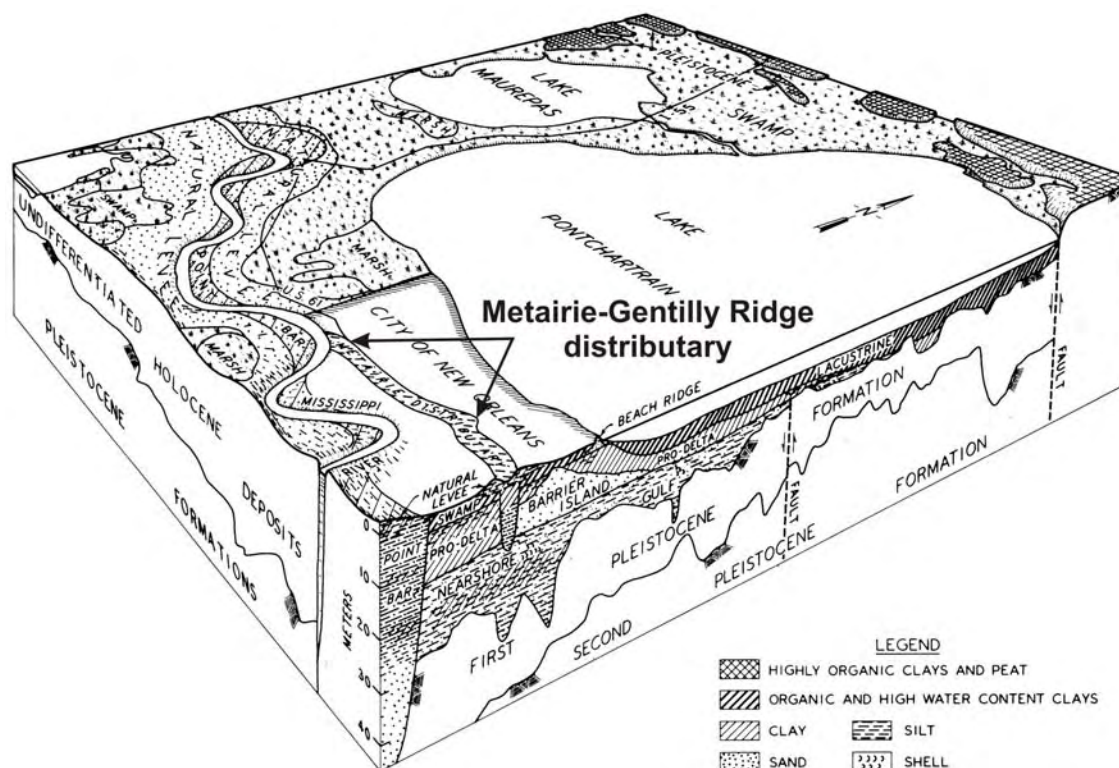


Figure 3.7: Block diagram of the geology underlying New Orleans (modified from Kolb and Saucier, 1982). The principal feature dividing New Orleans is the Metairie distributary channel, shown here, which extends to a depth of 50 feet below MGL and separates geologic regimes on either side. Note the underlying faults, especially that bounding the northern shore of Lake Pontchartrain.

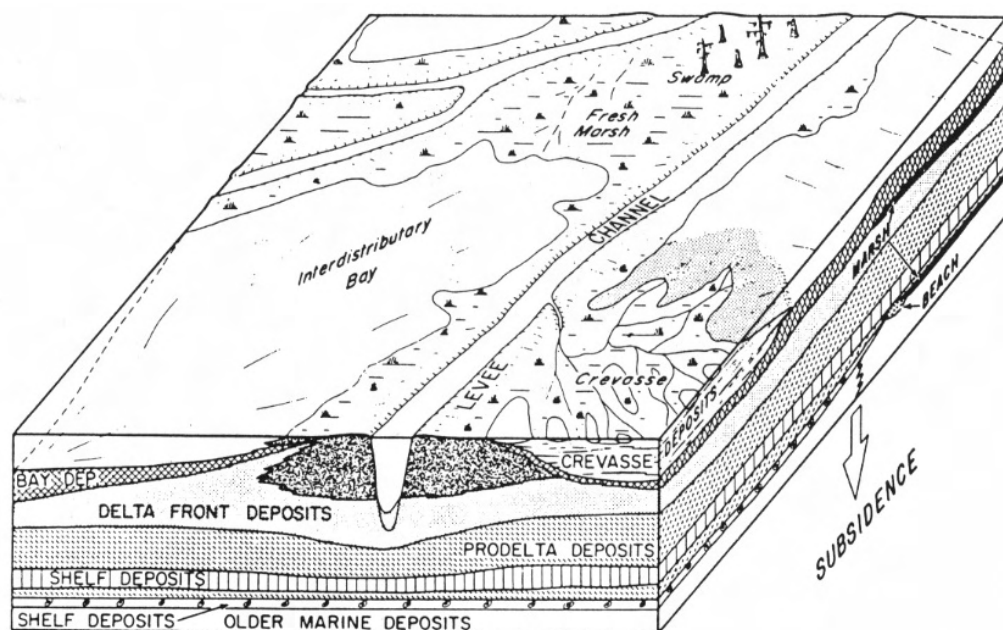


Figure 3.8: Block diagram illustrating relationships between subaerial and subaqueous deltaic environments in relation to a single distributary lobe (taken from Coleman and Roberts, 1991). The Lakeview and Gentilly neighborhoods of New Orleans are underlain by interdistributary sediments, overlain by peaty soils laid down by fresh marshes and cypress swamps.

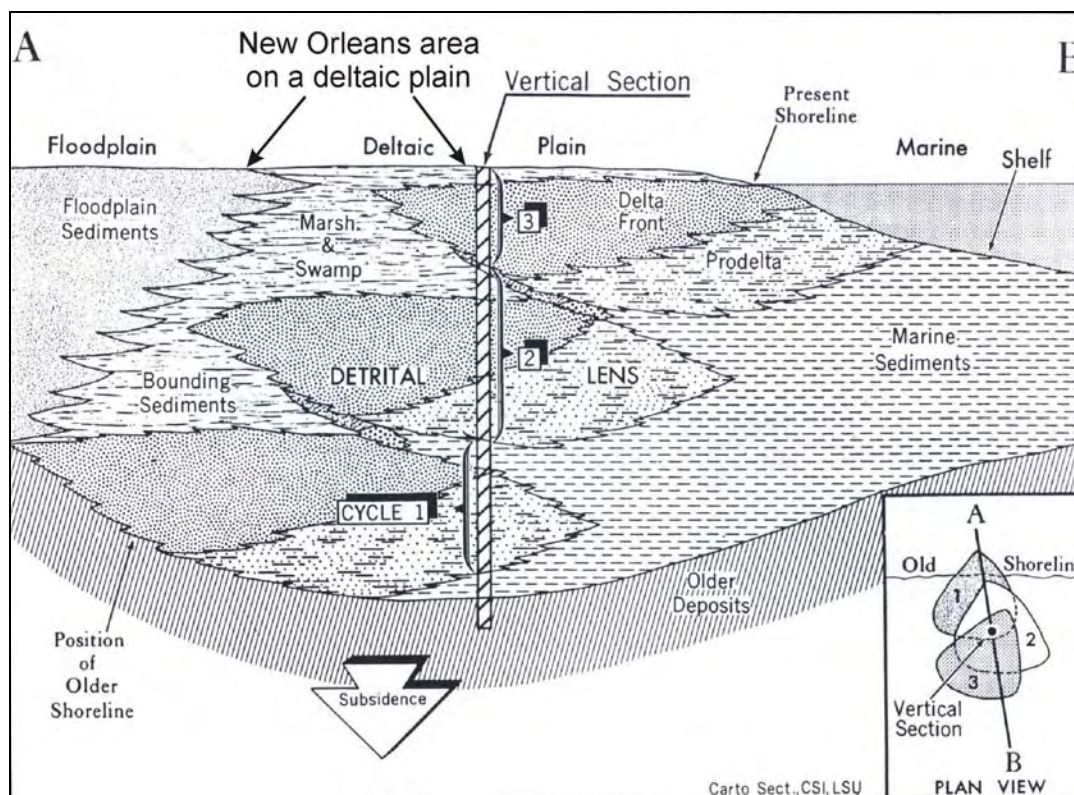


Figure 3.9: Sedimentary sequence caused by overlapping cycles of deltaic deposition, along a trend normal to that portrayed in the previous figure (modified from Coleman and Gagliano, 1964). As long as the distributary channel receives sediment, the river mouth progrades seaward. Lower New Orleans lies on a deltaic plain with marsh and swamp deposits underlying the Lakeview and Gentilly neighborhoods, and delta front deposits closer to Metairie-Gentilly Ridge, the nearest distributary channel.



Figure 3.10: Portion of the 1849 flood map showing the mapped demarcation between brackish and fresh water marshes along Lake Pontchartrain (taken from WPA-LA, 1937). This delineation is shown on many of the historic maps, dating back to 1749.

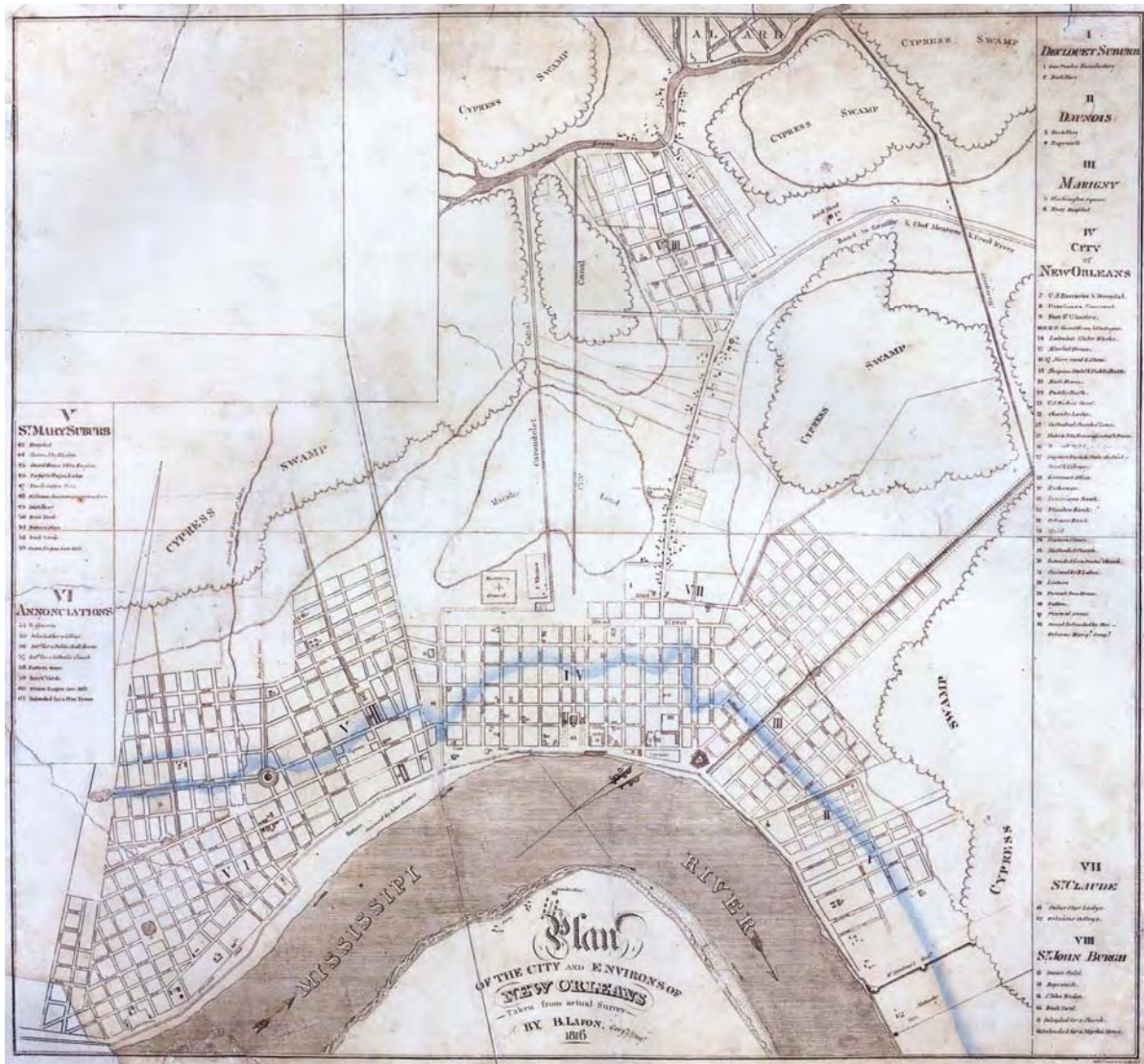
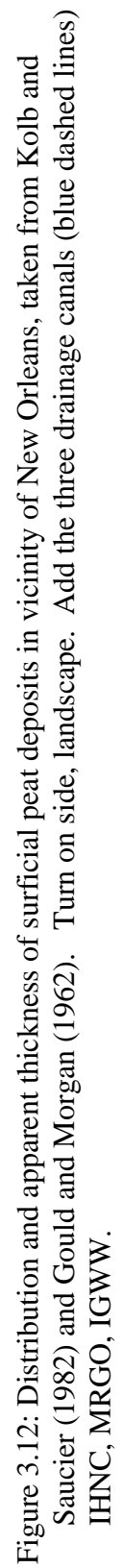


Figure 3.11: 1816 flood map of New Orleans showing areal distribution of cypress swamps north of the old French Quarter (from the Historic New Orleans Collection). These extended most of the distance to the Lake Pontchartrain shore.



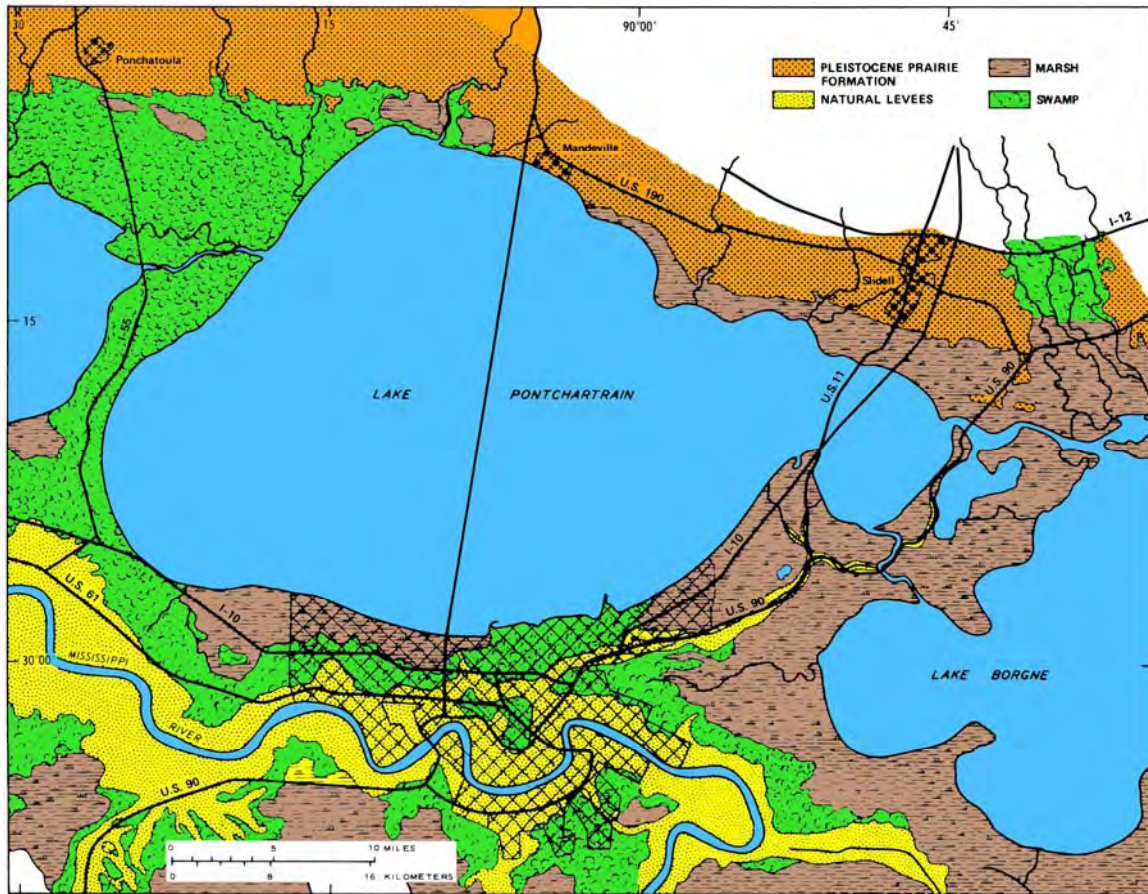


Figure 3.13: Geologic map of the greater New Orleans area, modified from Kolb and Saucier (1982). The sandy materials shown in yellow are natural levees, green areas denote old cypress swamps and brown areas are historic marshlands. The stippled zone indicates the urbanized portions of New Orleans.

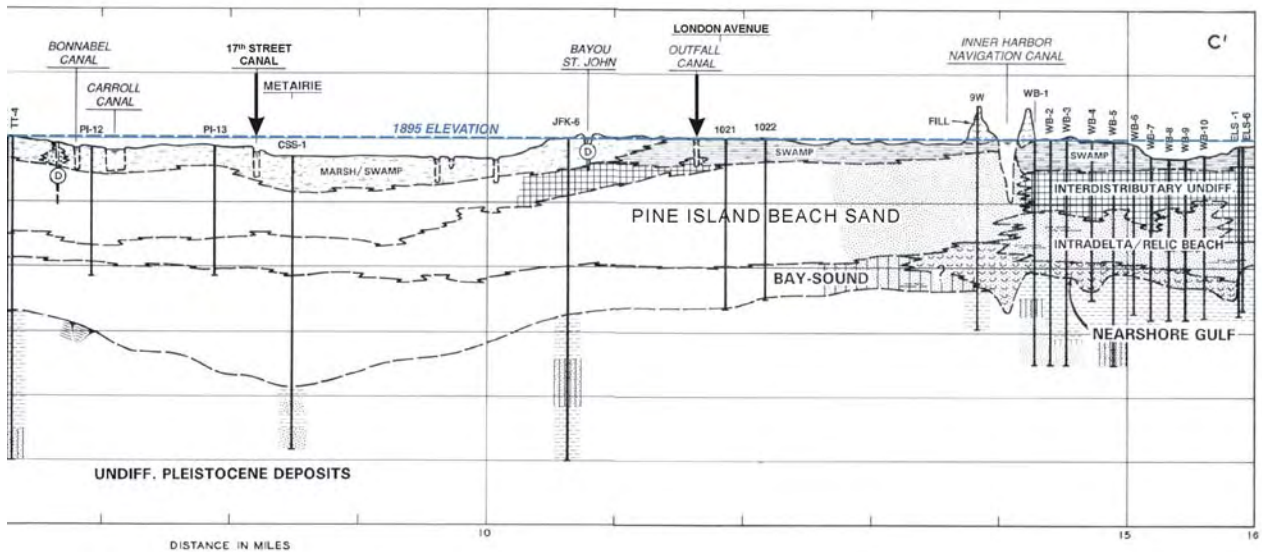


Figure 3.14: Geologic cross section along south shore of Lake Pontchartrain in the Lakeside, Gentilly, and Ninth Ward neighborhoods, where the 17th Street, London Avenue, and IHNC levees failed during Hurricane Katrina on Aug 29, 2005. Notice the apparent settlement that has occurred since the city survey of 1895 (blue line), and the correlation between settlement and non-beach sediment thickness. This east-west section was taken from Dunbar et al. (1994).



Figure 3.15: Wood and other organic debris was commonly sampled in exploratory borings carried out after Hurricane Katrina throughout the city. This core contains wood from the old cypress marsh that was recovered near the 17th Street (Metairie Relief) Canal breach. Organic materials are decaying throughout the city wherever the water table has been lowered, causing the land surface to subside (photo by C. M. Watkins).

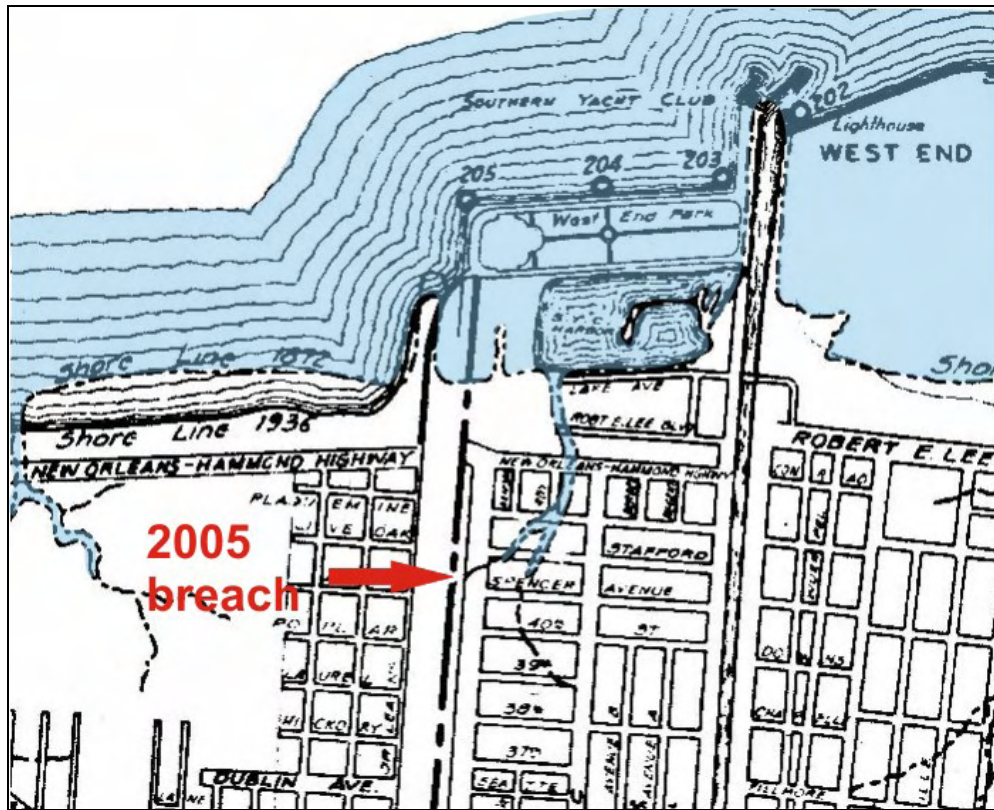


Figure 3.16: Overlay of 1872 map by Valery Sulakowski on the WPA-LA (1937) map, showing the 1872 shoreline and sloughs (in blue) along Lake Pontchartrain. Although subdivided, only a limited number of structures had been built in this area prior to 1946. The position of the 2005 breach along the east side of the 17th Street Canal is indicated by the red arrow.

17th St. OUTFALL CANAL LEVEE BREAK BORING AND CROSS SECTION LOCATIONS

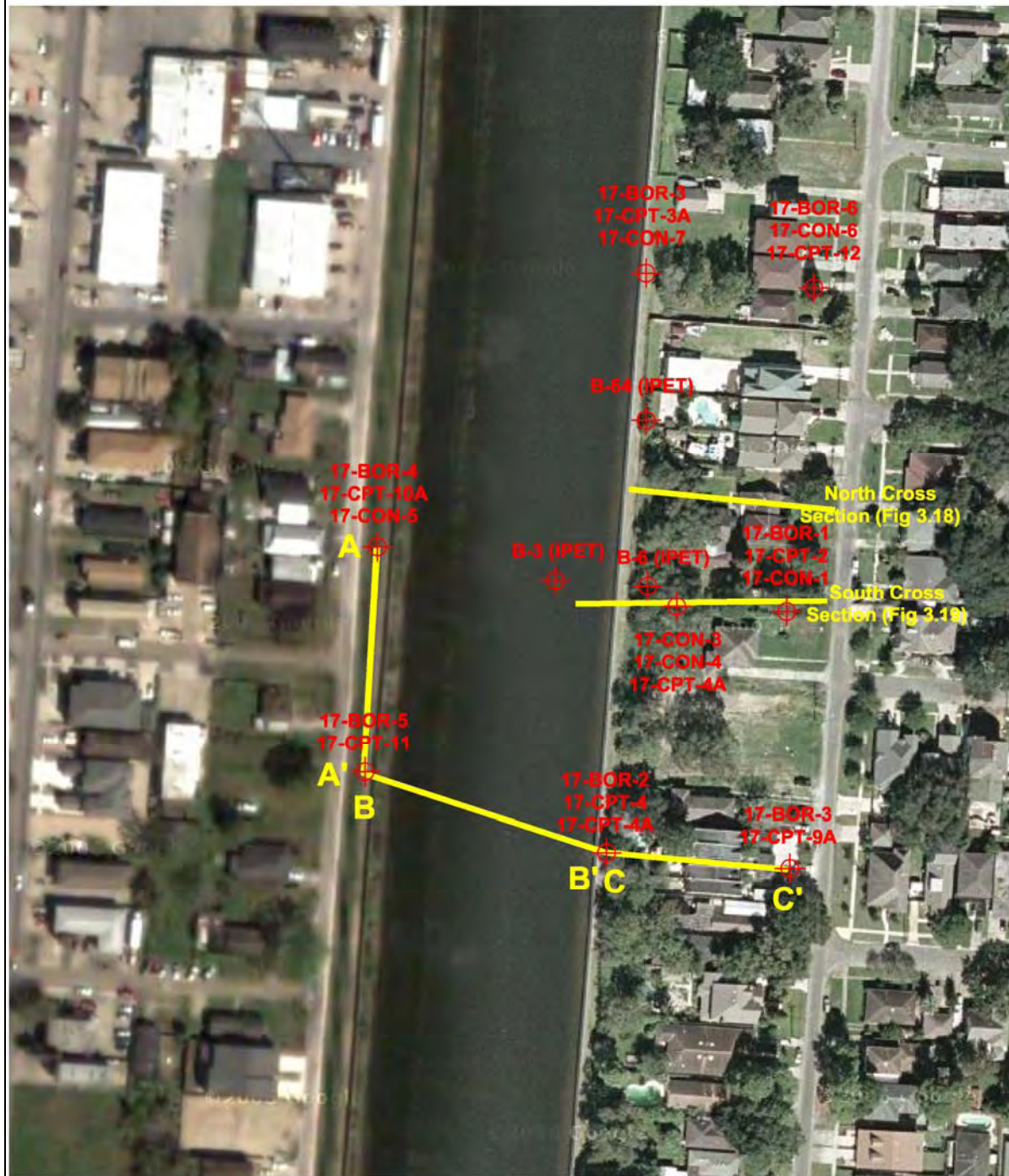


Figure 3.17: Aerial photo of the 17th Street Canal breach site before the failure of August 29, 2005. The yellow lines (at middle right) indicate the positions of the geologic sections presented in Figures 3.18 and 3.19, while cross sections A-C' are shown in Figure 3.21.

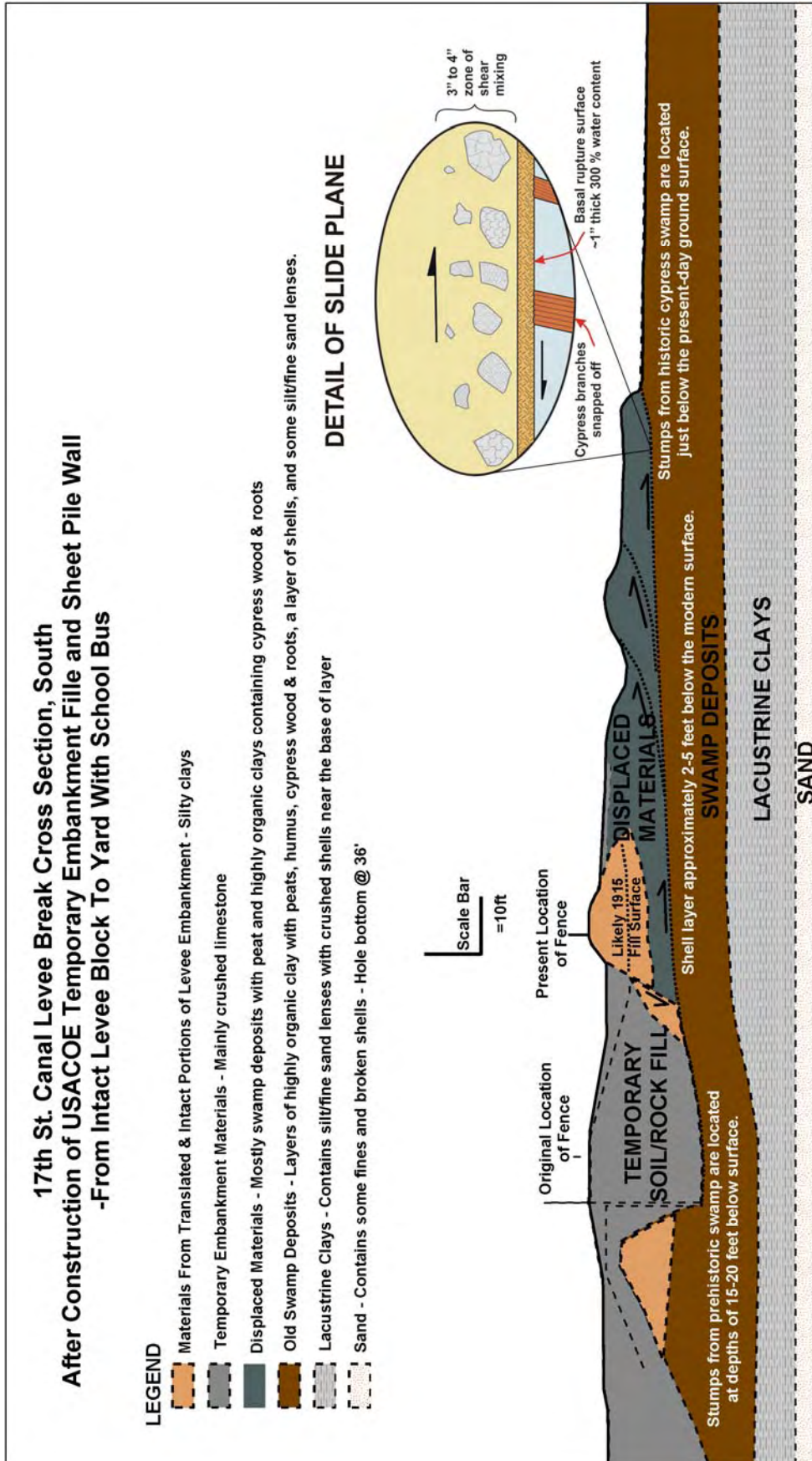


Figure 3.18: West-to-east geologic cross section through the 17th Street Canal failure approximately 60 feet north of the northern curb of Spencer Avenue, close to the yellow school bus. A detailed sketch of the basal rupture surface is sketched above right. The slip surface was about one inch thick with an extremely high moisture content (watery ooze). A zone of brecciation 3 to 4 inches thick was above this. Numerous pieces of cypress wood, up to 2 inches diameter, were sheared off along the basal rupture surface.

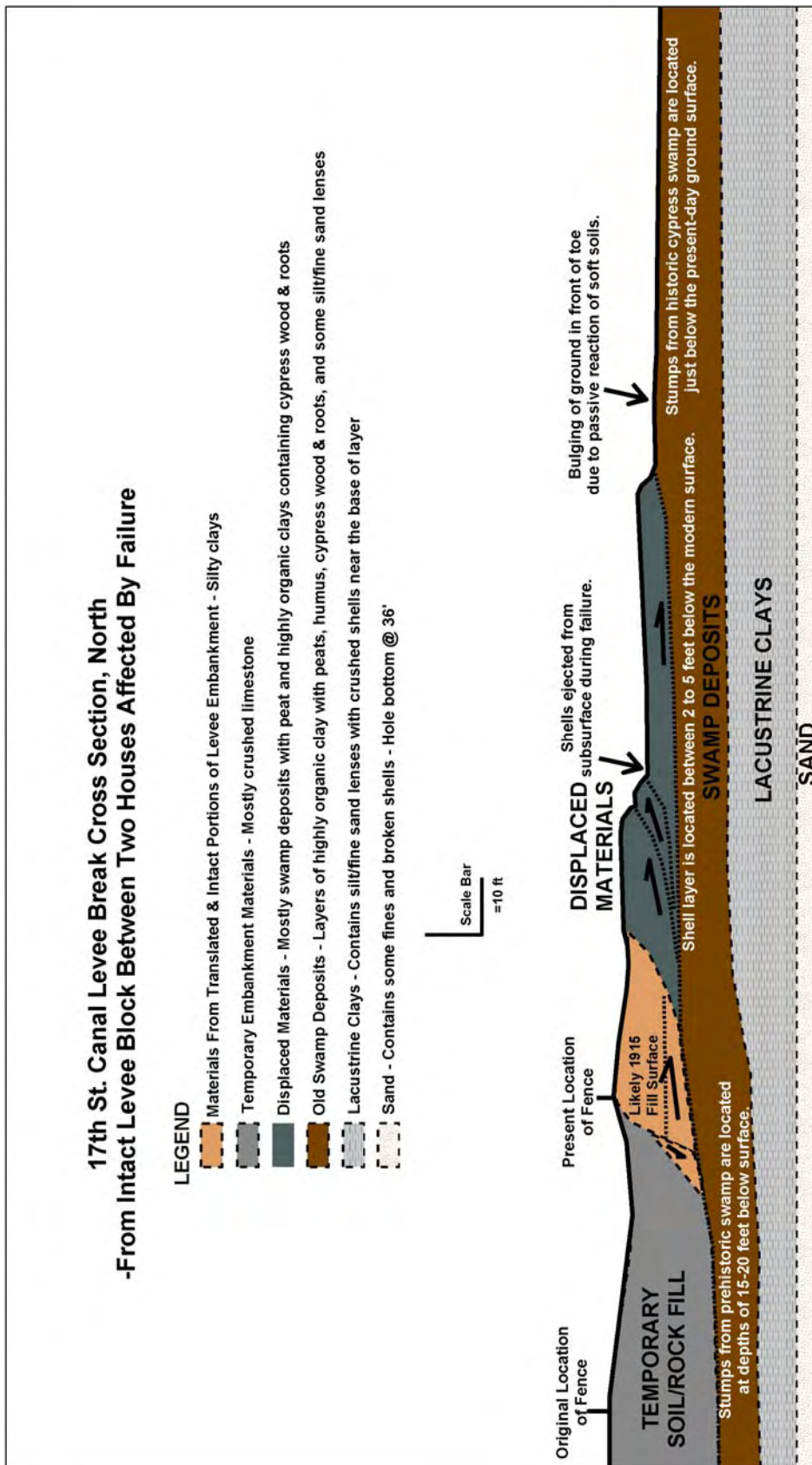


Figure 3.19: West-to-east geologic cross section through the 17th Street Canal failure approximately 140 feet north of the northern curb of Spencer Avenue, just north of the first surviving home next to the canal. Large quantities of bivalve shells were extruded by high water pressure along the regressing toe thrusts (shown in Figure 3.20). Note the slight back rotation of the distal thrust block.



Figure 3.20: Bivalve shells ejected by high pore pressures emanating from toe thrusts on landside of failed levee at the 17 Street Canal (detail view at upper left). These came from a distinctive horizon at a depth of 2 to 5 feet below the pre-failure grade (photo by C.M. Watkins).

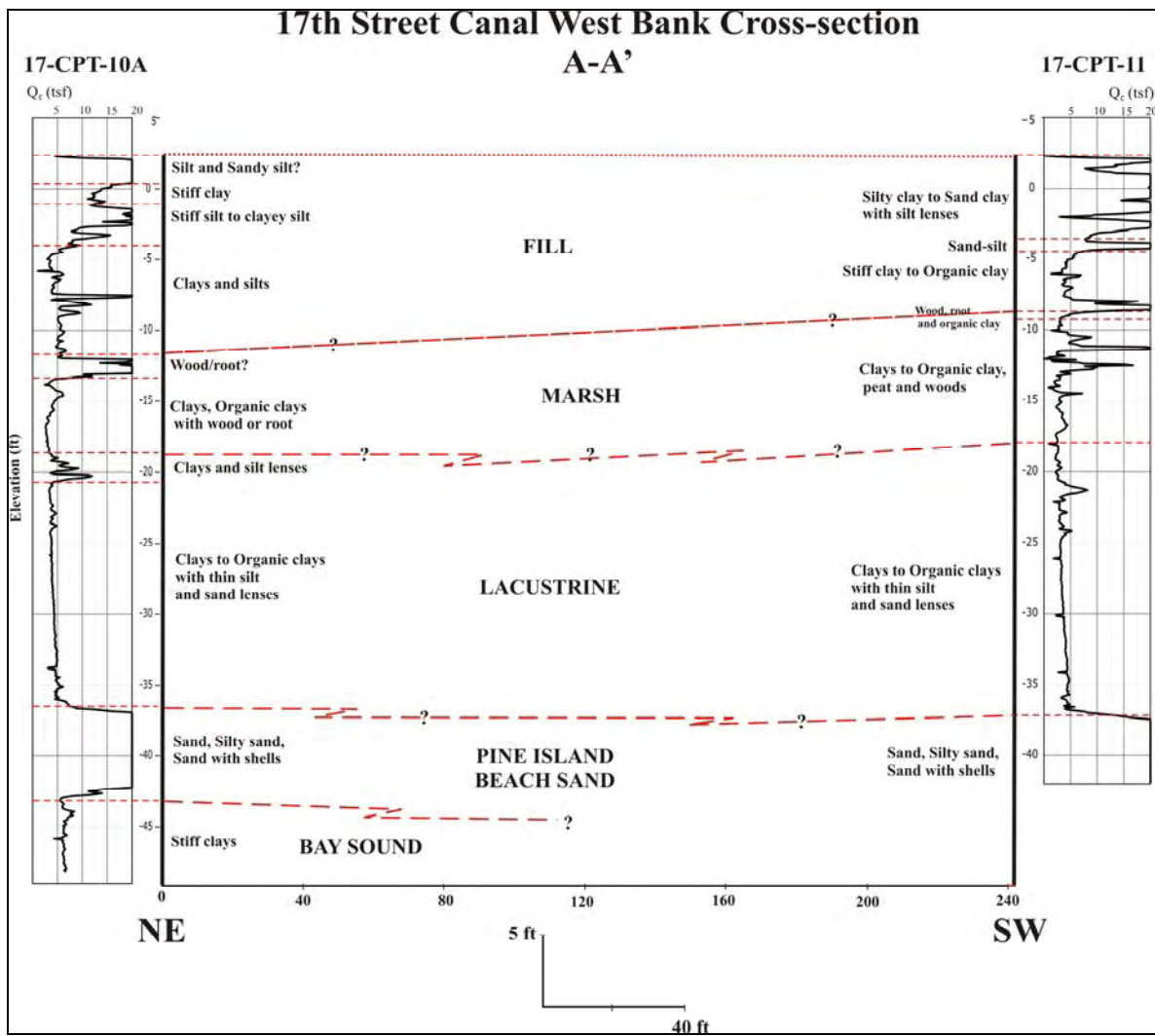


Figure 3.21 - upper: Stratigraphic interpretations between CPT soundings along western embankment of the 17th Canal (in Jefferson Parish), opposite the breach on the east side. The marsh-swamp deposits are dipping slightly towards Lake Pontchartrain, while the lacustrine clays appear to be flat lying.

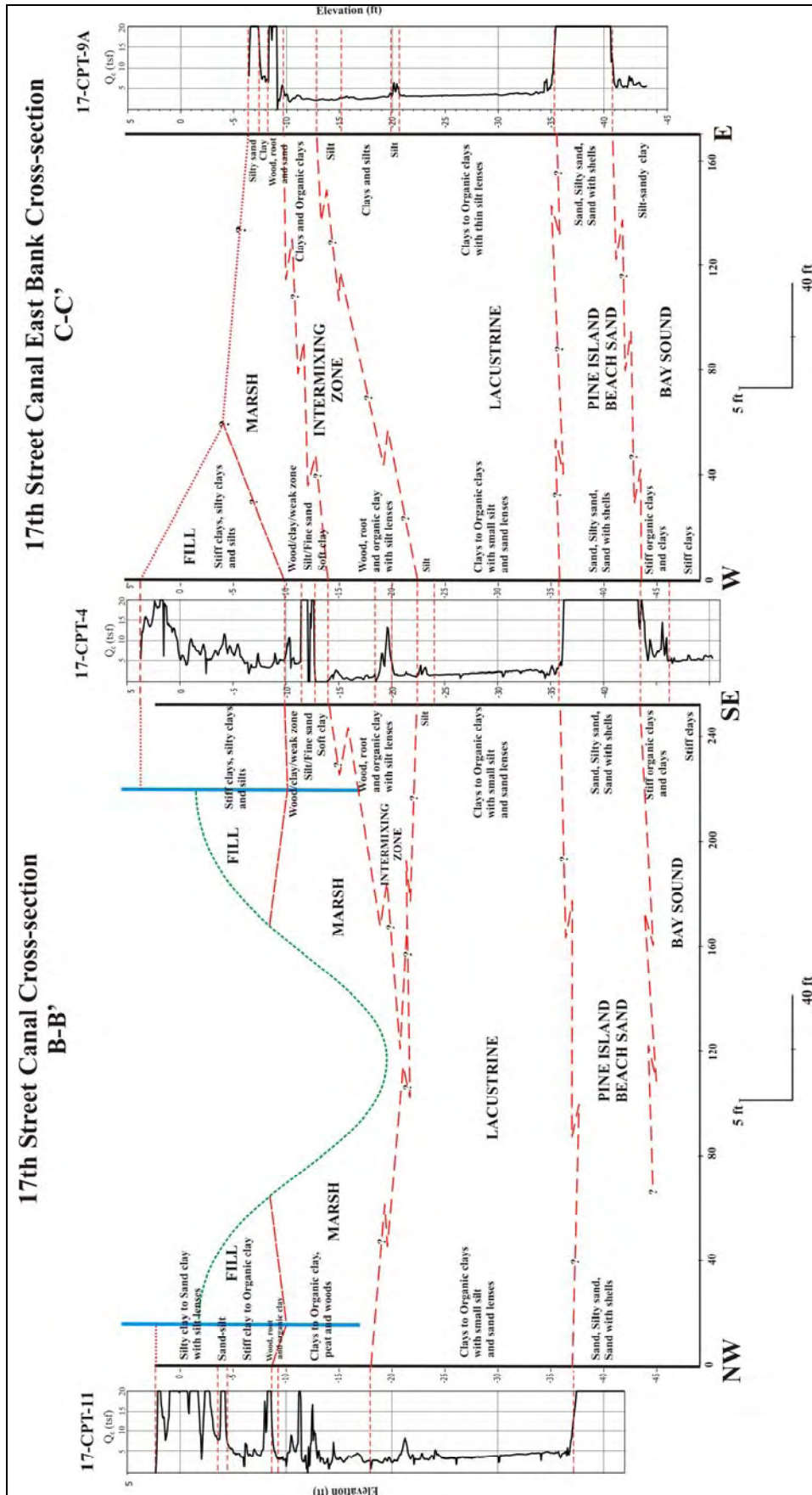


Figure 3.21-lower: Stratigraphic interpretations and cross-canal correlations in vicinity of the 17th Street Canal breach on August 29, 2005. The swamp much appeared to be thinning northerly, as does the underlying Pine Island Beach Trend. The lacustrine clays appear to thicken northward, as shown. The approximate positions of the flood walls (light blue) and canal bottom (dashed green) are indicated, based on information provided by the Corps of Engineers (IPET, 2006).

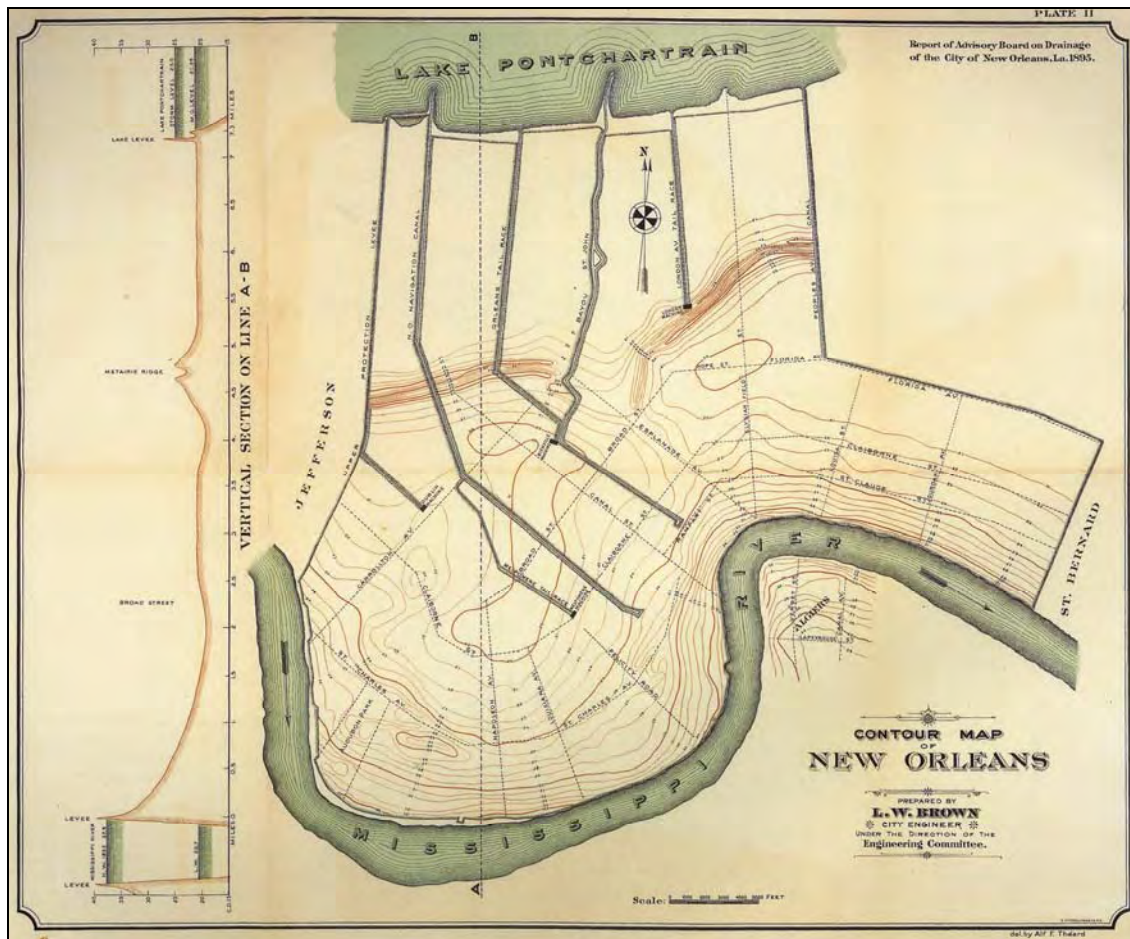


Figure 3.22: Topographic map with one foot contours prepared under the direction of New Orleans City Engineer L.W. Brown in 1895. This map was prepared using the Cairo Datum, which is 21.26 feet above Mean Gulf Level.

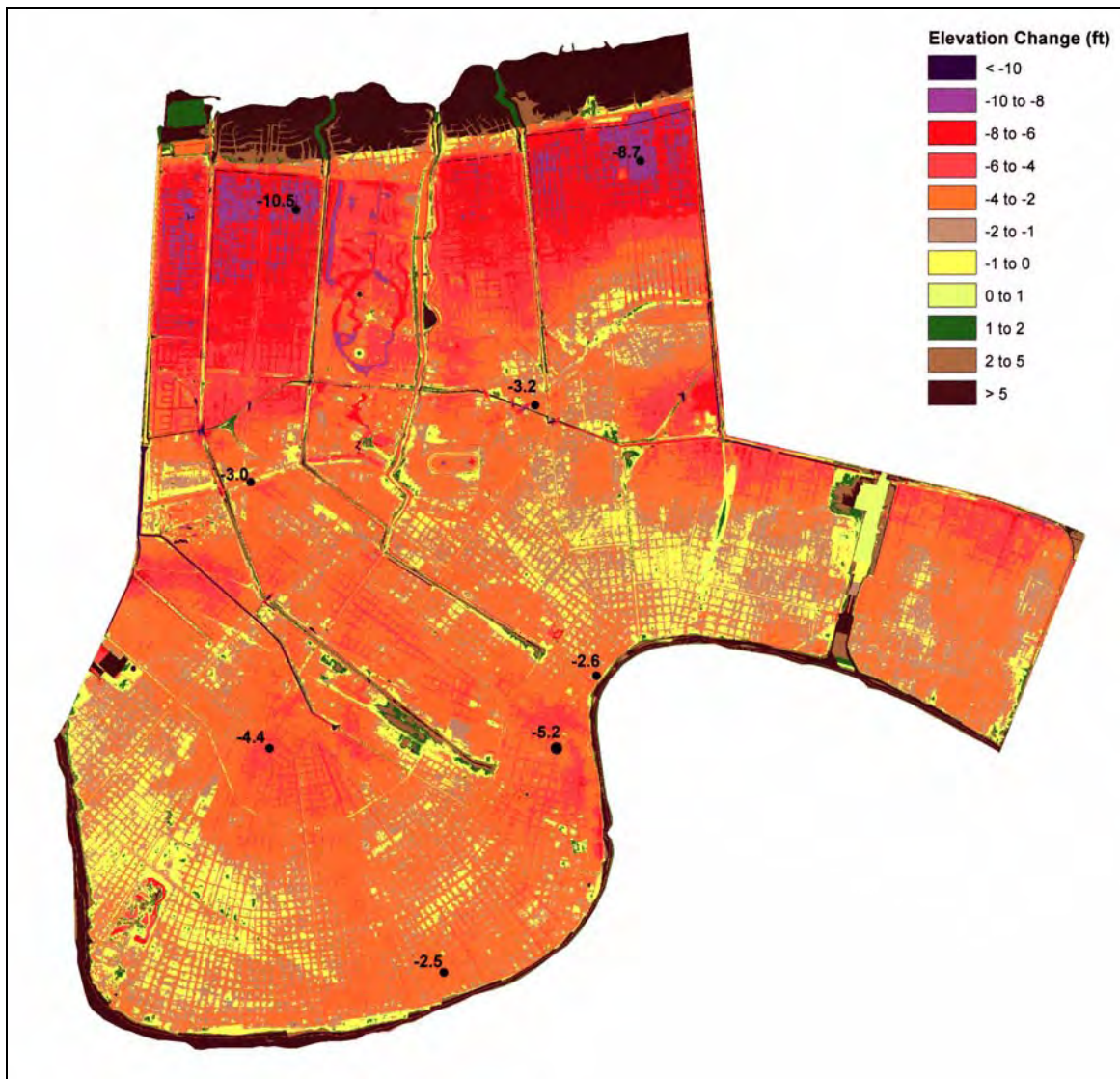


Figure 3.23: Map showing relative elevation change between 1895 and 1999/2002, taken from URS (2006). The approximate net subsidence was between 2 and 10+ feet, depending on location. The brown colored zones along Lake Pontchartrain and the Mississippi River are areas where substantive fill was placed during the same interim.

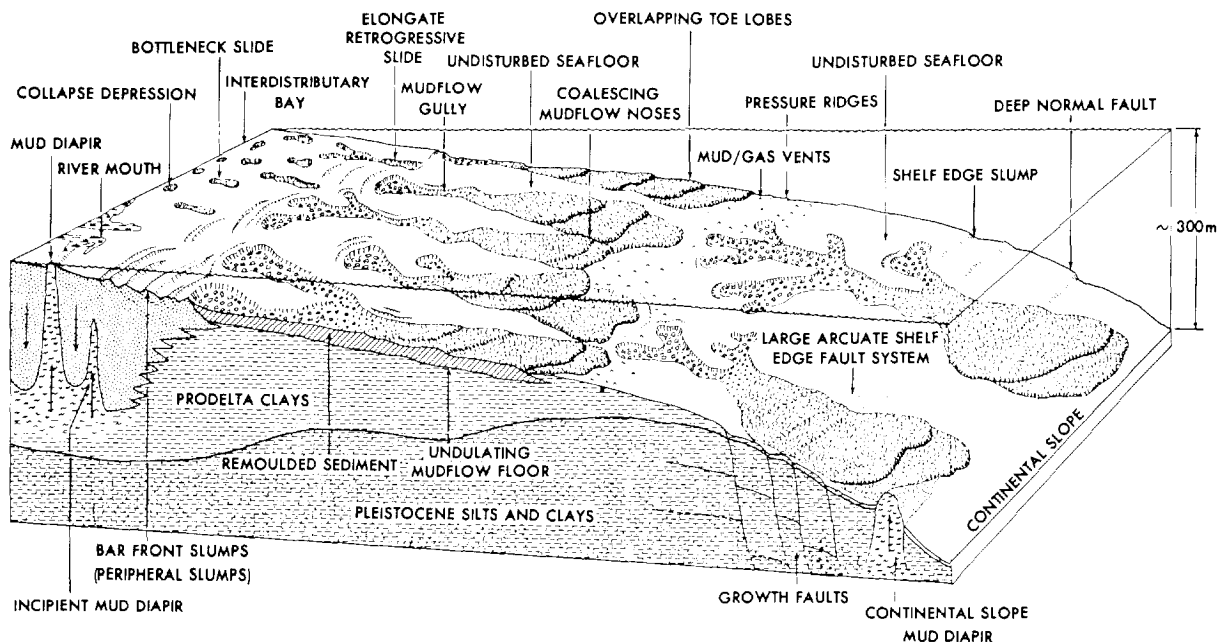


Figure 3.24: Block diagram illustrating various types of subaqueous sediment instabilities in the Mississippi River Delta, taken from Coleman (1988).

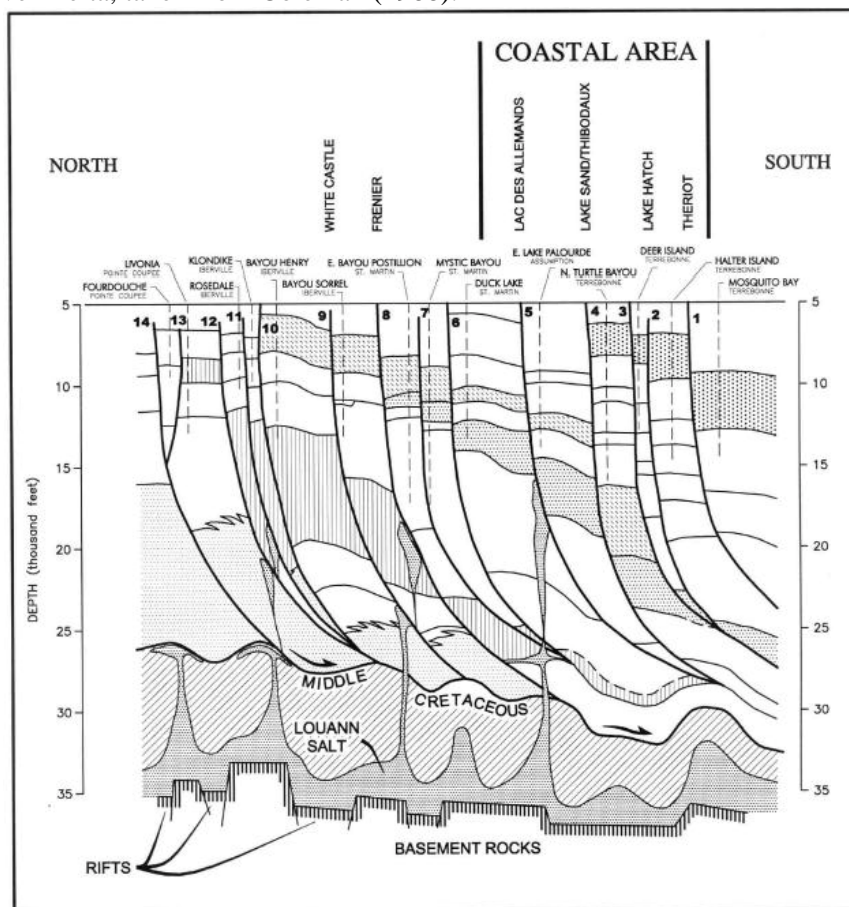


Figure 3.25: Geologic cross section through the Gulf Coast Salt Dome Basin, taken from Adams (1997). This shows the retrogressive character of young lystric normal faults cutting coastal Louisiana, from north to south. The faults foot in a basement-salt-decollement surface of middle Cretaceous age (> 100 Ma).

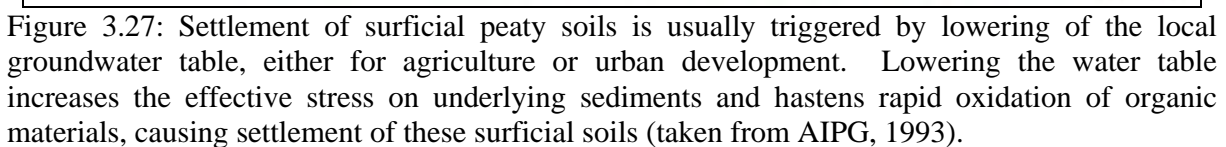
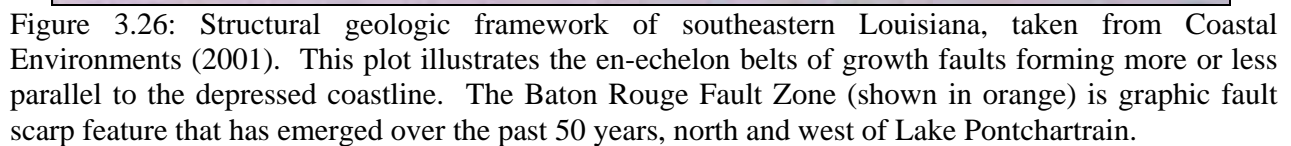




Figure 3.28: Upper photo shows gross near-surface settlement of homes in the Lakeview neighborhood, close to the 17th Street Canal breach. Most of the homes were constructed from 1956-75 and are founded on wood piles about 30 feet deep. The lower photo shows protrusion of a brick-lined manhole on Spencer Avenue, suggestive of at least 12 inches of near surface settlement during the same interim (photos by J. D. Rogers).

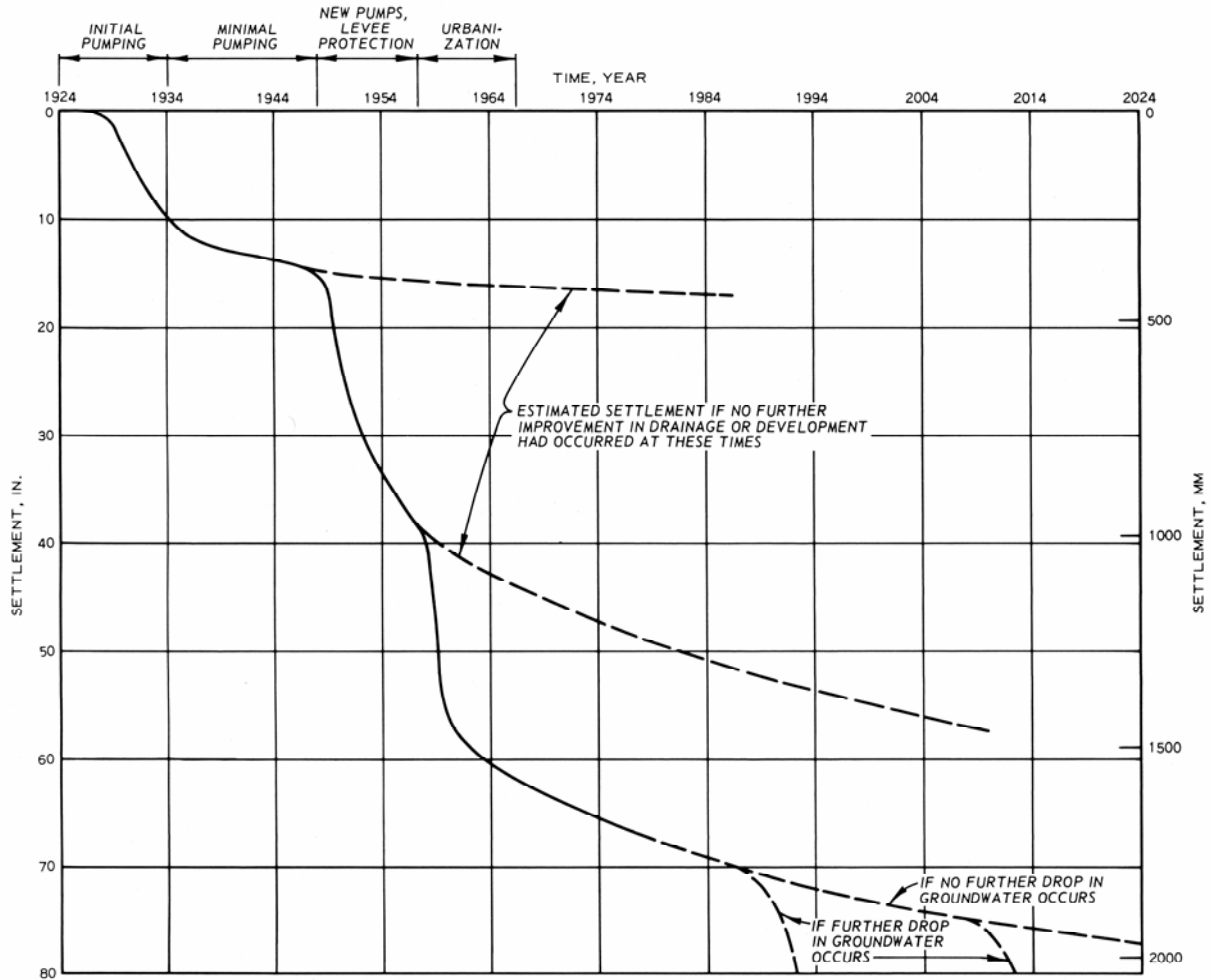


Figure 3.29 – Record of historic settlement in the town of Kenner, which is characterized by 6.5 to 8 feet of surficial peaty soils (taken from Kolb and Saucier, 1982). The earlier episodes of settlement were triggered by groundwater withdrawal (for industrial and municipal usage), while the later episode was caused by drainage associated with urban development. This area was covered by dense cypress swamps prior to development.

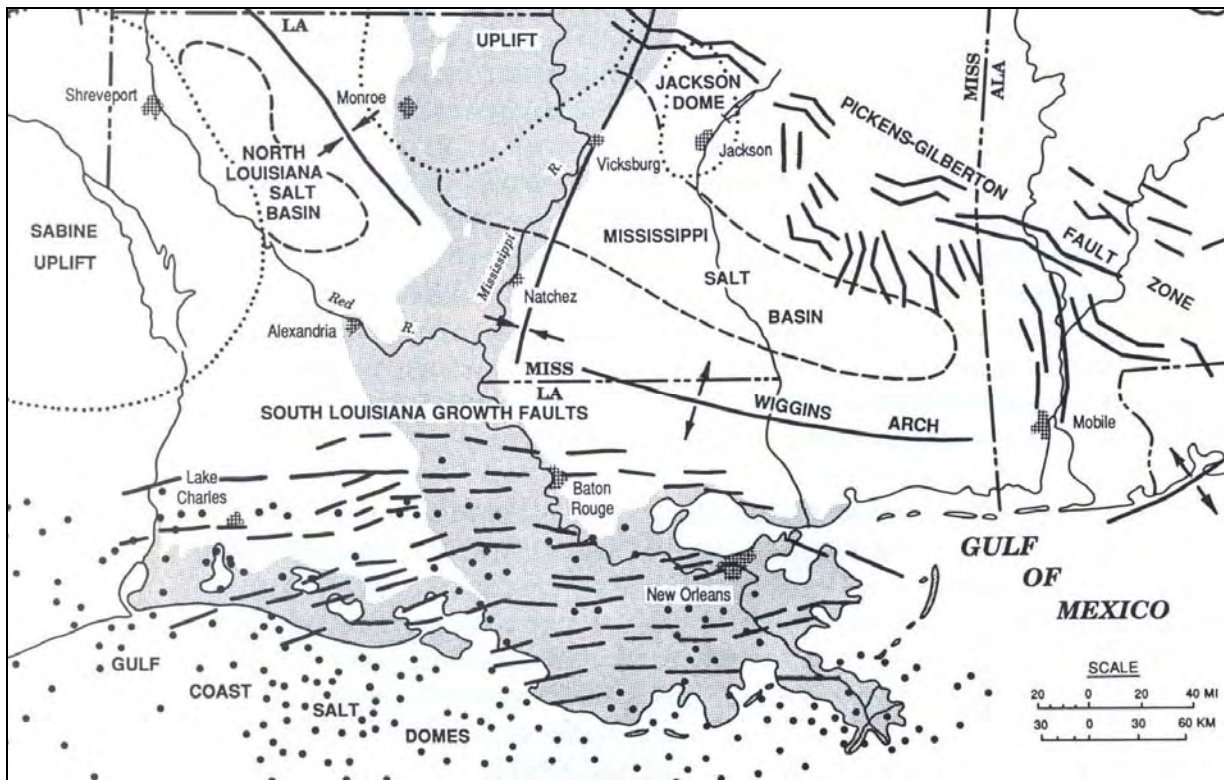


Figure 3.30: Structural geologic framework of the lower Mississippi River Delta, taken from Saucier (1994). Growth faults (solid black lines) perturb the coastal deltaic plain, as do salt domes (shown as dots). The nearest salt domes to New Orleans are 9 to 15 miles southwest of New Orleans. This study did not uncover evidence of growth faults materially affecting any of the levee failures from Hurricane Katrina, although such possibility exists.

CHAPTER FOUR: HISTORY OF THE NEW ORLEANS FLOOD PROTECTION SYSTEM

4.1 Origins of Lower New Orleans

New Orleans is a deep water port established in 1718 about 50 miles up the main stem of the Mississippi River, on the eastern flank of the Mississippi River Delta. New Orleans was established by the French in 1717-18 to guard the natural portage between the Mississippi River and Bayou St. John, leading to Lake Pontchartrain. The 1749 map of New Orleans by Francois Saucier noted the existence of fresh water versus brackish water swamps along the southern shore of Lake Pontchartrain.

The original settlement was laid out as 14 city blocks by 1721-23, with drainage ditches around each block. The original town was surrounded by a defensive bastion in the classic French style. The first levee along the left bank of the Mississippi River was allegedly erected in 1718, but this has never been confirmed (it is not indicated on the 1723 map reproduced in Lemmon, Magill and Wiese, 2003). New Orleans' early history was typified by natural catastrophes. More than 100,000 residents succumbed to yellow fever between 1718 and 1878. Most of the city burned to the ground in 1788, and again, in 1794, within sight of the largest river in North America. The settlement was also prone to periodic flooding by the Mississippi River (between April and August), and flooding and wind damage from hurricanes between June and October. Added to this was abysmally poor drainage, created by unfavorable topography, lying just a few feet above sea level on the deltaic plain of the Mississippi River, which is settling at a rate of between 2 and 10 feet (ft) per century.

The tendency for flooding during late spring and summer runoff came to characterize the settlement. The natural swamps north of the original city were referred to as "back swamps" in the oldest maps, and "cypress swamps" on maps made after 1816. During the steamboat era (post 1810), New Orleans emerged as the major trans-shipment center for river-borne to sea-born commerce, vice-versa, and as a major port of immigration. By 1875 it was the 9th largest American port, shipping 7,000 tons annually. In 1880, after completion of the Mississippi River jetties (in 1879), New Orleans experienced a 65-fold increase in seaborne commerce, shipping 450,000 tons, jumping it to the second largest port in America (New York then being the largest). New Orleans would retain its #2 position until well after the Second World War, when Los Angeles-Long Beach emerged as the largest port, largely on the strength of its container traffic from the Far East. New Orleans remains the nation's busiest port for bulk goods, such as wheat, rice, corn, soy, and cement.

New Orleans has always been a high maintenance city for drainage. The city's residential district did not stray much beyond the old Mississippi River levee mound until after 1895, when serious attempts to bolster the Lake Pontchartrain "back levee" and establish a meaningful system of drainage were undertaken by the city. Most of the lowland cypress swampland between Mid-Town and Lake Pontchartrain was subdivided between 1900-1914, after the City established and funded a Drainage Advisory Board to prepare ambitious plans for keeping New Orleans dry all the way to Lake Pontchartrain's shoreline. This real estate

bonanza increased the City's urban acreage by 700% and their assessed property values by 80% during the same interim (Campanella, 2002). Most of these lots were developed after the First World War (1917-18). Another 1,800 acres was reclaimed from the south shore of Lake Pontchartrain in 1928-31, between the mouth of the 17th Street Canal on the west and the Inner Harbor Navigation Canal (IHNC) on the east. The entire area was subsequently built-out following the Second World War, between 1945-70.

4.2 Mississippi River Floods

The Mississippi River drains 41% of the continental United States, with a watershed area of around 1,245,000 square miles (mi²), according to the U.S. Army Corps of Engineers. This makes it the third largest watershed of any river in the world. Although its official length is 2,552 miles (if measured from Itasca State Park in Minnesota), when combined with the Missouri River (2,540 miles long), it is the longest river in North America, with a combined length of 3,895 miles. Prior to 1950, the sediment load (suspended and dissolved) transported by the Mississippi River averaged between 550 and 750 million tons per annum (Meade and Parker, 1985). Since 1950, the average annual suspended discharge of the river has decreased to 220 million tons/yr (Meade and Parker, 1985), because of the construction of dams and maintenance of the navigation channel (which includes dredging). The Mississippi River now ranks as the 6th largest silt load in the world.

The Mississippi's flood plain upstream of Baton Rouge is an alluvial valley that, prior to 1928, was periodically subject to inundation by flooding. Vast tracts of the flood plain were periodically inundated. 26,000 square miles of land (mi²) was inundated during the 1927 flood; 20,312 mi² in the 1973 flood, and 15,600 mi² in the 1993 flood (which focused on the lower Missouri watershed). 75% of the sediment deposited on the North American continent is overbank flood plain silt, which spills onto the flood plain when floods spill over natural or man-made levees. At its widest point in the Yazoo Basin, the Mississippi flood plain is more than 80 miles wide.

4.2.1 Mississippi River is the High Ground

The river is the high ground in the Mississippi Embayment (Figure 4.1). A vexing problem with a high silt load river is that it tends to build up its own bed, which prevents drainage of the adjoining flood plains. Sediment is deposited on the adjoining lowlands when the river spills up out of its channel during flood stage. Sediments are hydraulically sorted during this process, becoming increasingly fine-grained and soft with increasing distance from the river channel, as sketched in Figure 4.2. Millions of acres of flood plain swamps and marshlands in the Mississippi Embayment downstream of Gape Girardeau, MO were reclaimed by mechanically excavated drainage ditches, beginning around 1910, when large rail-mounted dragline excavators became available. This machinery was also employed for levee construction on the MR&T Project (after 1928) as well as drainage work for agricultural reclamation.

4.2.2 Flooding from the Mississippi River

A great number of floods have occurred in the lower Mississippi Valley during historic time, including: 1718, 1735, 1770, 1782, 1785, 1971, 1796, 1799, 1809, 1811, 1813,

1815, 1816, 1823, 1824, 1828, 1844, 1849, 1850, 1851, 1858, 1859, 1882, 1892, 1893, 1903, 1907, 1908, 1912, 1913, 1916, 1920, 1922, 1923, 1927, 1929, 1932, 1936, 1937, 1945, 1950, 1957, 1958, 1973, 1974, 1975, 1979, 1983, 1984, 1993, and 1997.

The most damaging to New Orleans were those in: 1816, 1826, 1833, 1849, 1857, 1867, 1871, 1874, 1882, 1884, 1890, 1892, 1893, 1897, 1903, 1912, 1913, 1922, 1927, 1937, 1947, 1965, 1973, 1979, 1993, and 2005. But, the last flood of any consequence to affect the City of New Orleans emanating from the Mississippi River was in 1859!

New Orleans was founded in 1718. In April 1719 the town's founder Jean Baptiste le Moyne, Sieur de Bienville, reported that water from the Mississippi River was regularly inundating the new settlement with half a foot of water. He suggested constructing levees and drainage canals, and soon required such drainage work of all the landowners. In 1734-35 the Mississippi River remained high from December to June, breaking levees and inundating the settlement.

Flood protection from the Mississippi River was originally afforded by heightening of the river's natural bank overflow levees (Hewson, 1870), like those shown in Figure 4.3. Crevasses, or crevasse-splays, (Figure 3.8) are radiating tensile cracks that form in the bank of a river, natural levee, man-made levee, or drainage canal. Crevasse-splays are often triggered by underseepage along preferential flow conduits, such as old sand-filled channels or the radiating distributaries of previous channel breaks. For these reasons, crevasse-splays often occur at the same locations repeatedly.

On May 5, 1816 the Mississippi levee protecting New Orleans gave way at the McCarty Plantation, in present-day Carrollton, and within a few day water filled the back portion of the city, extending from St. Charles Avenue to Canal and Decatur Streets, flooding the French Quarter. The water was only drained after a new drainage trench was excavated through Metairie Ridge and channels connecting to Bayou St. John.

On May 4, 1849 the Mississippi River broke the levee at the Suavé Plantation at River Ridge, 15 miles upstream of New Orleans. Within four days this water reached the New Basin Canal, and within 17 days was flooding the French Quarter in New Orleans proper, flooding the area down slope (north of) of Bienville and Dauphine Streets. The 1849 flood waters rose at an average rate of one foot every 36 hours, which allowed residents ample time to evacuate. Uptown residents thought about severing the levee along the New Basin Canal to prevent water levels building up on their side, but those living on the opposite side of the canal threatened to prevent such measures using armed force. Shortly thereafter the New Basin upper levee collapsed, diverting flood waters to Bayou St. John and thence, into Lake Pontchartrain. A nine foot deep lake developed in what is now the City's Broadmoor area, flooding 220 city blocks and necessitating the evacuation of 12,000 residents.

The 1849 crevasse at Suavé Plantation was eventually plugged by driving a line of timber piles and piling up thousands of sand bags against these on the land-side of the pile wall. This work was of unprecedented proportions until that time and took six weeks to complete before the river's waters were once again confined to their natural channel. Drainage trenches were then excavated through Metairie [distributary] Ridge to channel ponded water out to Lake Pontchartrain. By mid-June 1849 the water was finally receding and

residents began re-entering their flooded homes, spreading lime to combat mold, mildew, and impurities.

Between 1849 and 1882, four major crevasse-splays occurred at Bonnet Carré, on the eastern bank of the Mississippi River, about 33 river miles upstream of New Orleans. The Bonnet Carré crevasses left a large fan-shaped imprint on the landscape. In fact, during the flood of 1849, a 7,000-foot-wide crevasse developed at Bonnet Carré which diverted flow from the Mississippi into Lake Pontchartrain for more than six months. This breach had to be filled so sufficient discharge could flow down the main channel to allow ocean going vessels to reach New Orleans.

The 1849 floods were the last time that the eastern bank of the Mississippi River was breached affecting New Orleans proper. In 1858 high water lapped over the east bank levee, but this was followed a few days later by a break on the west bank of the river (at Bell Plantation), which drew down the high water threatening New Orleans. The Bell Plantation crevasse remained open for six months. In 1859 the rear portion of New Orleans again flooded, between Carrollton and Esplanade Avenues, flooding one-third of the City between January and March.

The City of New Orleans and the Mississippi River became important battlegrounds during the American Civil War between 1861-65. Early in the conflict a principal goal of the Union forces west of the Appalachian Mountains was to sever the Confederacy along the Mississippi River. Union forces had a distinct advantage insofar as they retained most of their naval power, allowing them to blockade Confederate ports. General Ulysses Grant achieved considerable notoriety for his early campaigns up the Cumberland and Tennessee Rivers, and later, in the successful siege of Vicksburg, which gave northern forces control of the Mississippi, isolating 40,000 Confederate troops west of the river, where they played no further significant role in the conflict. Grant recognized the pivotal military role of the great river, because it was his Army's vital supply line. Grant turned to his engineers on numerous occasions and ordered the construction of cutoffs (Figure 4.4), some of which were successful, while others, such as that a short distance downstream of Vicksburg, were not.

The success or failure of the man-made cutoffs depended on a number of factors, such as time of year, severity of the spring flood, and ability to meter flows into the cutoffs trying to control the erosion caused by dropping the water over oversteepened gradients. These experiences were drawn upon soon after the war (Hewson, 1870) to create an inland empire through drainage of low lying swamps and construction of thousands of miles of privately constructed levees to keep the river from flooding reclaimed tracts.

During the post Civil War boom that witnessed significant reclamation of flood-prone tracts in the Mississippi flood plain, a pattern of protection emerged as the established cities like New Orleans battled the Mississippi: that being of adjacent breaks, upstream at Bonnet Carré and downstream, in Plaquemines Parish, often providing "safety valves" that reduced high water in the river along the New Orleans waterfront. The western bank would breach again in 1893, at the Ames Plantation in Marrero. Breaks in adjoining areas gradually gave rise to rumors about levees being purposefully undermined to save the more valuable property within the city, which reached epic proportions during the record flood of 1927, when the

levees adjoining Plaquemines Parish were dynamited no less than seven times, by City officials worried that their own protective works would crumble and give way (Barry, 1997).

Army Engineer A.A. Humphreys and civilian engineer Charles Ellet were funded by Congress in separate contracts to make a scientific examination of the Mississippi River in 1850. Ellet completed his work in 1851, but Humphreys did not complete his report until 1861, after suffering a nervous breakdown (Barry, 1997). Humphreys exerted significant control of the Mississippi River as Chief of the Army Corps of Engineers between 1866-1879. He was the father of the Corps' flawed "levees only policy" of flood control, which remained in effect till the 1927 flood, which triggered the creation of the Jadwin Plan, embodied in the Federal Flood Control Act of 1928 (Morgan, 1971; Shallat, 1994). The "levees only policy" maintained that the Mississippi River could be constrained within its natural low flow channel by extending its natural levees upward, assuming the channel would downcut its bed vertically during high flows, thus remaining in an artificially confined channel. This logic was hopelessly flawed in that it ignored the river's serpentine curvature, which causes it to loop on itself in a seemingly endless series of "meander belts" across the floodplain. Because of this curvature, the channel is seldom symmetrical (as portrayed in Figure 4.2), but generally exhibits marked asymmetry, like that shown in Figure 4.5.

In 1871 the Mississippi River once again spilled its eastern bank at Bonnet Carré, 33 miles upstream of New Orleans. The massive break diverted much of the river's flow into Lake Pontchartrain, raising its level. A strong north wind pushed lake water up into the Metairie and Gentilly ridges, filling the then-existing system of drainage canals. A levee on the Hagan Avenue (now the Jefferson Davis Parkway) drainage canal gave way, flooding the back side of New Orleans, including the Charity Hospital, a town landmark.

The 1927 flood was the largest ever recorded on the lower Mississippi Valley (Figure 4.6). The deluge was preceded by a record 18 inches of rain falling on New Orleans in a 48 hour period in late March 1927, which was followed by six months of flooding. The levees that were supposed to protect the valley broke in 246 places, inundating 27,000 square miles of bottom land; displacing 700,000 people, killing 1,000 more (246 in the New Orleans area), and damaging or destroying 137,000 structures.

There was an enormous public outcry for the government to do something more substantive about flood control. Fearing the worst, the political leadership of New Orleans sought relief by dynamiting the Mississippi levee in Plaquemines Parish, downstream of New Orleans. By the time promises were made regarding damage compensation and the necessary permission was granted, the flood had crested and begun to subside. No less than seven sequences of dynamiting ensued, all promoted by fear. The initial dynamiting of the Caernarvon levee below New Orleans with 30 tons of dynamite devastated much of St. Bernard and Plaquemines Parishes, and their residents were never remunerated in any meaningful way for their damages. The saddest aspect of the dynamiting was that it was unnecessary, as several levees gave way upstream of New Orleans, one the very afternoon of the dynamiting, and the river level at New Orleans never regained its maximum crest during the remainder of that record year (Barry, 1997).

4.3 The Mississippi River and Tributaries Project 1931-1972

The Corps of Engineers' Mississippi River & Tributaries (MR&T) Project was authorized by Congress in the Flood Control Act of 1928, which emanated from the Great Flood of 1927 on the lower Mississippi River. At the time of its introduction it was referred to as The "Jadwin Plan," because Major General Edgar Jadwin was the Army's Chief of Engineers at the time it was issued, on December 1, 1927 (Jadwin, 1928). It was incorporated into the Federal Flood Control Act of May 15, 1928, which authorized \$325 million to the Mississippi River Commission (created in 1879) controlled by the Corps of Engineers to provide for flood protection along the Mississippi River between Cape Girardeau, MO and Head-of-Passes, LA. In essence the Mississippi River Commission adopted the Mississippi River & Tributaries Project, and the commission's responsibilities, annual budget, expenditures and importance increased by an order of magnitude, where it remains more-or-less today. Actual construction did not begin until 1931, when the authorized funds were finally appropriated by Congress.

The original flood control plan selected a project flood of 2,360,000 cubic feet per second (cfs) at the mouth of the Arkansas River and 3,030,000 cfs at the mouth of the Red River. These figures were about 11% greater than the record 1927 flood at the junction of the Mississippi and Arkansas Rivers and 29% greater than 1927 flood at the junction of the Mississippi and Red Rivers, 60 miles downstream of Natchez, MS.

The Jadwin Plan proposed four major elements to control the flow of the Mississippi River. These were: 1) levees to contain flood flows wherever practicable, or necessary to avoid razing large sections of existing cities and transportation infrastructure; 2) bypass floodways to accept excess flows of the river, passing these into relatively undeveloped agricultural basins or lakes; 3) channel improvements intended to stabilize river banks, to enhance slope stability and commercial navigation; and 4) improvements to tributary basins, wherever possible. This category included dams for flood storage reservoirs, pumping plants, and auxiliary channels.

The main stem levees (Figure 4.7) were intended to protect the Mississippi alluvial valley against flooding by confining the river to its low flow channel. The main stem, or so-called "federal levees," extend 1,607 miles along the Mississippi River, with another 600 miles along the banks of the lower Arkansas, Red, and Atchafalaya Rivers.

A vexing problem with maintaining 1,552 miles of flood control levees in the lower Mississippi Valley has been the complex and ever-changing foundations upon which they are founded (Figure 4.7). In addition, channel curvature promotes undercutting of the outboard banks of bends, often depositing these materials in semi-linear stretches of channel a short distance downstream, because of lower gradients. This sediment reduces freeboard and raises flow levels, often beyond design assumptions. Crevasses are often sand-filled distributary channels that form preferred seepage paths beneath the flood plain during high flow. These high permeability corridors lie beneath earthen levees like ticking time bombs, waiting to explode (areas indicated by red arrows on Figure 4.8).

The 1928 Jadwin Plan also sought to emplace storage facilities wherever practicable in the four principal watersheds bordering the lower Mississippi Valley: the St. Francis Basin in

southeastern Missouri and northeastern Arkansas; the Yazoo Basin in northwestern Mississippi; the Tensas Basin in northeastern Louisiana; and the Atchafalaya Basin in southern Louisiana. Five flood control reservoirs were constructed in these basins as part of the MR&T Project: Wappapello Dam and Reservoir in the St. Francis Basin; and four dams in the Yazoo Basin: Arkabutla, Sardis, Enid, and Grenada.

Bypass floodways were constructed by the Corps of Engineers. These included: 1) the Birds Point-New Madrid Floodway between Cairo, IL and New Madrid, MO (which depends on a fuse plug levee in lieu of a spillway; only used once, in 1937); 2) The Old River or Red River Landing Diversion structure, intended to divert half the project flood (1,500,000 cfs) from the main channel into the Atchafalaya River through the Morganza and West Atchafalaya floodways; 3) The Bonnet Carré bypass and floodway, a concrete spillway capable of diverting 250,000 cfs into Lake Pontchartrain during periods of high flow, about 30 miles upstream of New Orleans. The locations of MR&T structures in close proximity to New Orleans are shown on Figure 4.9.

This was followed by numerous channel improvements and stabilization measures which have been implemented as needed along the entire course of the navigable river channel, to enhance river bank stability and commercial navigation. The Corps typically employs channel cutoffs to shorten the river channel and increase hydraulic grades, which reduces flood heights. They employ armored revetments to retard channel migration and meandering. Countless dikes have been employed to direct the river's flow, beneath the channel surface. Annual dredging is required to maintain navigable channels, as sediment is deposited by seasonal high flows. These activities have combined to reduce the annual sediment yield of the river by 60% (Kesel, 2003).

The Bonnet Carré bypass and Old River Control Structure (into the Atchafalaya Basin) are major elements of the MR&T Project that protect New Orleans from a Mississippi River flood by reducing the volume of flow that passes the city. The Bonnet Carré spillway was the first structural element of the MR&T Project to be constructed, in 1931, and initially used during the 1937 flood. It is opened up whenever the river level exceeds 19.0 to 19.6 ft in New Orleans and can draft off 250,000 cfs into Lake Pontchartrain.

The Old River Control Structure was not authorized by Congress until 1954. It was intended to draft off 600,000 cfs of the Mississippi's flow during an extreme flood event and prevent capture of the Mississippi River by the Atchafalaya River, which would have occurred naturally by 1975 (because the flow distance of the Atchafalaya to the Gulf of Mexico is only one-third the distance taken by the present channel of the Mississippi; see Fisk, 1952 and McPhee, 1989). The Corps constructed the Old River Control Structure and lock from 1961-63. The project was intended to divert 30% of the Mississippi River Project Flood into the Atchafalaya Basin. The Old River Control Structure has only been used once, during the Flood of 1973, when it nearly failed catastrophically (MRC, 1975; Noble, 1976). In the wake of this failure, the capacity was doubled with construction of an auxiliary structure, completed by the Corps of Engineers in 1986, doubling the bypass capacity at Old River into the Atchafalaya-Morganza Basin to 1,220,000 cfs.

The average height of the MR&T levees above the natural levees in the Gulf Region is about 16 ft (Kolb and Saucier, 1982). The crest of the flood protection levee along the eastern

bank of the Mississippi River is 24.5 feet MGL at Carrollton in New Orleans, as shown in Figure 4.10. The Lake Pontchartrain protection levee varies between 13.5 and 18.0 feet MSL, as shown in Figure 2.6, 2.7, and 4.10. All of the neighborhoods north of Metairie Ridge lie below sea level. The worst flooding scenario for New Orleans would be a breach of the Mississippi River levee because of its elevated position, which would engender rapid erosion and high spill velocities, which could overwhelm the City's lowest neighborhoods before residents could effect an escape.

From its inception, the 1928 Flood Control Act has been modified every few years by additional authorizations from Congress, usually based on modifications requested by the Corps of Engineers. These included expenditures for establishment of an emergency fund for maintenance and rescue work (1930) and acquisition of lands for floodways, etc. These early changes resulted in the Flood Control (Overton Act) Act of 1936, which established a national flood control policy to be administered by the Corps of Engineers, beyond the lower Mississippi Valley. Even with these sweeping changes, more acts followed in quick succession throughout the late 1930s and 1940s (for instance, a 1937 act authorized \$52 million for strengthening of levees following the disastrous 1937 flood in the Ohio and Mississippi Valleys). This pattern of amended flood control acts and authorized expenditures continued throughout the 1940s, 50s, 60s, and 70s, usually following flood years.

Today, 3,714 miles of flood control levees have been authorized for construction under the Mississippi River & Tributaries Project. 3,410 miles of levees have been completed and 2,786 miles are in place to grade and section. On the main stem of the Mississippi River, 1,602 miles of levees have been completed. Work on the main stem levees of the Mississippi River is approximately 89 percent complete and work on tributary levees is approximately 75 percent complete.

4.3.1 Dimensions of Navigation Channels Maintained by the Corps of Engineers on the Lower Mississippi River

Over the next 60 years Congress added new river borne transport projects, extending up the Mississippi drainage and elsewhere, creating an intricate system of barge commerce that demands constant maintenance, clearing, patching, and dredging. In addition to ensuring flood protection, the Corps of Engineers was also charged with maintaining year-round navigation for the Port of New Orleans, which was the nation's second largest port facility when MR&T project work commenced in 1931.

After the mouths of the Mississippi River had been opened and maintained in a navigable state (the first jetty was completed in 1879), navigation interests lobbied Congress to establish and maintain "feeder" channels to the Mississippi River and deepen the main stem channel to accommodate more modern vessels, with deeper draft. In 1945 Congress authorized the development of a navigation channel for oceangoing traffic in the lower reaches of the Mississippi River. Over the past 60 years this system has been expanded greatly through a series of Congressional acts, until today it consists of 12,350 miles of navigable inland waterways. The depths and widths of the Mississippi River channel between Baton Rouge and the Gulf of Mexico have been established as:

- Baton Rouge to New Orleans - 40 by 500 feet
- Port of New Orleans - 35 by 1,500 feet, with portion 40 by 500 feet
- New Orleans to Head of Passes - 40 by 1,000 feet
- In Southwest Pass - 40 by 800 feet
- In Southwest Pass Bar Channel - 40 by 600 feet
- In South Pass - 30 by 450 feet
- In South Pass Bar Channel - 30 by 600 feet
- Mississippi River-Gulf Outlet - 36 by 500 feet
- Mississippi River-Gulf Outlet Bar Channel - 38 by 600 feet

4.4 Flooding of the New Orleans Area by Hurricanes

Hurricanes strike the Louisiana Coast with a mean frequency of two every three years (Kolb and Saucier, 1982). Since 1759, 172 hurricanes have struck southern Louisiana (Shallat, 2000). Of these, 38 have caused flooding in New Orleans, usually via Lake Pontchartrain. Some of the more notable events have included: 1812, 1831, 1860, 1893, 1915, 1940, 1947, 1965, 1969, and 2005.

In 1722 a hurricane destroyed most of embryonic New Orleans and raised the river by 8 feet. Had the river not been running low prior to the storm, the river might have overtopped its banks by as much as 15 feet. In 1778, 1779, 1780 and 1794 hurricanes struck the New Orleans area destroying many buildings and sinking ships. The worst storm of the early years was “The Great Louisiana Hurricane” of August 9, 1812. It rolled over the barrier islands and drowned Plaquemines and St. Bernard Parishes and the area around Barataria Bay under 15 feet of water. The parade ground at Fort St. Phillip was inundated by 8 feet of water and the shoreline along Lake Pontchartrain was similarly inundated, though this was far enough below the French Quarter to spare any flooding of the City.

The back side of New Orleans was afforded some natural protection by the Metairie, Gentilly, and Esplanade Ridges, which are recent distributary channels of the Mississippi River. These “ridges” were originally about 4 feet higher than the surrounding marshland, but much of the former cypress swamps and marshes (comprised of compressible peaty soils) have settled as much as 10 feet over the past 110 years, while the ridges, being underlain by sand, have only settled 1 to 2 feet. The ridges performed as quasi flood protection levees from storm surges emanating from Lake Pontchartrain during hurricanes. But the ridge also prevented drainage from moving between the old French Quarter and Lake Pontchartrain. The Carondelet, or Old Basin, canal was excavated between Basin Street and Bayou St. John, which formed the one low point between the elevated Metairie and Gentilly Ridge channels. The Old Basin Canal drained the French Quarter and allowed smaller craft to transit through the ridge to Lake Pontchartrain.

In June 1821 easterly winds surged off Lake Pontchartrain and pushed up Bayou St. John, flooding fishing villages and spilling into North Rampart Street until the winds abated and allowed the water to drain back into the lake. It was an ominous portent of things to come.

On August 16, 1831 “The Great Barbados Hurricane” careened across the Caribbean, striking the Louisiana coast west of New Orleans. The area south of town was again inundated by storm surge, while a three foot surge entered the city from Lake Pontchartrain. The Mississippi levee at St. Louis Street gave way, flooding the French Quarter. Heavy rains accompanying this storm added to the flooding and boats were the only means of moving about for several days.

Southeastern Louisiana suffered through three hurricanes during the summer and fall of 1860. On August 8th a fast moving hurricane swept 20 feet of water into Plaquemines Parish. The third hurricane struck on October 2nd making landfall west of New Orleans. It inundated Plaquemines, St. Bernard, and Barataria, causing a significant storm surge in Lake Pontchartrain which destroyed 20 lakeside settlements, washing out a portion of the New Orleans and Jackson Great Northern Railroad. Surge from this storm overtopped the banks along the Old and New Basin drainage canals and a levee along Bayou St. John gave way, allowing the onrushing water to flood a broad area extending across the back side of New Orleans.

Between 1860 and 1871 the city avoided serious flooding problems caused by hurricanes. In 1871 three hurricanes caused localized flooding, which proved difficult to drain. Flooding emanating from storm surges on Lake Pontchartrain during these storms overtopped the Hagen Avenue drainage canal between Bayou St. John and New [Basin] Canal, spilling flood waters into the Mid-City area. City Engineer W. H. Bell warned the city officials about the potential dangers posed by the drainage canals leading to Lake Pontchartrain, because the Mid-City area lay slightly below sea level (as seen on the 1895 Brown map in Figure 3.22).

The record hurricane of October 2, 1893 passed south of New Orleans and generated winds of 100 mph and a storm surge of 13 feet, which drowned more than 2,000 people in Jefferson Parish, completely destroying the settlements on the barrier island of Cheniere Caminada. This represented the greatest loss of life ascribable to any natural disaster in the United States up until that time. Seven years later, in August 1900, a hurricane passed directly over Galveston, TX, demolishing that city and killing between 6,000 and 8,000 people, which remains the deadliest natural disaster in American history. Prior to impacting Galveston, that hurricane tracked westerly parallel to the Gulf Coast about 150 miles south of New Orleans. Its flood surges were noted along the Gulf Coast, including Lake Pontchartrain’s south shore (Cline, 1926).

Prior to Katrina’s landfall in 2005, the most damaging hurricane to impact New Orleans was the Grand Isle Hurricane of September 29, 1915, a Category 4 event which produced winds as great as 140 miles per hour (mph) at Grand Isle. It slowed as it made landfall and eventually passed over Audubon Park, seriously damaging structures across New Orleans. Electrical power was knocked out, preventing the City’s new pumps from functioning. The wave crest height on Lake Pontchartrain rose to 13 ft, easily overtopping 6-foot high shoreline levee, destroying the lakefront villages of Bucktown (at end of 17th Street Canal), West End, Spanish Fort, and Lakeview (these lakeside settlements were swallowed up by the infilling of the Lake Ponchartain shoreline in 1928-31). The drainage canals were also overtopped, flooding the city behind Claiborne, leaving Mid-City and Canal Street under

several feet of water. This storm overwhelmed the City's defenses so quickly that 275 people were killed, mostly in the Lake Pontchartrain shoreline zone.

On September 19, 1947 an unnamed hurricane made landfall near the Chandeleur Islands, producing wind gusts between 90 and 125 mph, with 1 minute maximum of 110 mph. A storm surge of 9.8 ft reached Shell Beach on Lake Borgne. The runways at Moisant Airport were covered by 2 ft of water while Jefferson Parish was flooded to depths of 3+ ft. Sewage from an overwhelmed S&WB treatment plant stagnated in some of the drainage canals, producing sulfuric acid fumes that caused staining of lead-based paint on some of the homes in the Lakeview area, leaving them with unsightly black blotches. 51 people drowned and New Orleans suffered more than \$100 million in damages. City officials were unable to clear floodwaters through the drainage canals in the Lakeview, Gentilly, and Metairie neighborhoods for nearly two weeks. This was the first significant hurricane to strike New Orleans which generated a large body of reliable storm surge data, which was subsequently used in design of flood protection works by the Corps of Engineers (Figure 4.11). The New Orleans Times-Picayune prepared a map that showed reported depths and locations of flooding in the 1947 hurricane.

After the 1947 storm, hurricane protection levees were heightened along the south shore of Lake Pontchartrain and extended westward, across Jefferson Parish (constructed in 1949). In addition, the embankments along the old drainage canals were raised by earthfill to protect the Orleans and Jefferson Parishes from future storm surges off Lake Pontchartrain. The precise height of these additions depended on position and historic settlement up till that time. The entire Lakeview area north of what is now Interstate 610 (excluding the area filled by the Lakefront Improvement Project) was already more than -2 ft below sea level by the late 1930s (WPA-LA, 1937).

Hurricane Betsy was a fast moving storm that made landfall at Grand Isle, LA on September 9-10, 1965. Wind meters at Grand Isle recorded gusts of up to 160 mph and a 15.7 ft storm surge that overwhelmed the entire island. Winds gusts up to 125 mph were recorded in New Orleans along with a storm surge of 9.8 ft, which overwhelmed both sides of the Inner Harbor Navigation Canal (IHNC), flooding the Ninth Ward, Gentilly, Lake Forest, and St. Bernard Parish areas (Figure 4.12), as well as all of Plaquemines Parish, causing the worst flooding since 1947, and revealing inadequacies in the levee protection system surrounding the city. 81 people were killed by the storm (58 in Louisiana), which was the first natural catastrophe in America to exceed \$1 billion in damages (USACE, 1965). Damage in southeast Louisiana totaled \$1.4 billion, with \$90 million of that being to New Orleans.

In October 1965 Congress approved a \$2.2 billion public works bill that included \$250 million for Louisiana projects and \$85 million down payment for a system of levees and barriers around New Orleans (Figure 4.13). This work included raising the Lake Pontchartrain levee to a height of 12 ft above Mean Gulf Level (MGL) in response to the flooding caused by Betsy. The Orleans Levee Board also let contracts to pound steel sheetpile walls along the crests of their drainage canal levees to increase their effective height, so storm surges on Lake Pontchartrain would not overtop the drainage canals (which had occurred in 1915, 1947, and 1965, but without catastrophic loss of the canal levees). The uncased sheetpiles were intended to be a temporary measure, awaiting a permanent solution

that envisioned placement of concrete flood walls using the sheetpiles as their foundations, funded by the Federal government. These short-term improvements spared the city from similar flooding in 1969 when Hurricane Camille struck the area.

Prior to Katrina, the only other Category 5 hurricane to make landfall on the United States was Hurricane Camille in August 1969 (the atmospheric pressure on landfall was second only to the Labor Day Hurricane of 1935). Camille made landfall on August 17th, its eye crossing the Mississippi Coast at Pass Christian, about 52 miles east northeast of New Orleans. Wind velocities in the eye of the storm reached 190 mph, while gusts on land exceeded 200 mph, causing most wind meters to fail (the highest recorded gust was 175 mph). Camille annihilated the coastal communities between Henderson Point and Biloxi, and caused extensive flooding of 3,900 mi² of coastal lowland between lower Plaquemines Parish and Perdido Pass, AL. The peak storm surge measured 25 feet above MGL near Pass Christian, MS (a record), 15 ft in Boothville, LA, 9 ft in The Rigolets, and 6 ft in Mandeville, LA. The death toll from Camille was 258 people, with 135 of these being from the Mississippi coast (9 were killed in Louisiana). 73,000 families either lost homes or experienced severe damage and the official damage toll was \$1.4 billion, with damages in Louisiana totaling \$350 million. A particularly vexing aspect of Camille was that it occurred just four years after Hurricane Betsy, which had been touted as something between a 1-in-200 to 1-in-300 year recurrence frequency event (USACE, 1965).

On September 28, 1998 Hurricane Georges wrecked havoc across the Caribbean, pummeling Haiti, the Dominican Republic, Puerto Rico and other islands. Georges appeared to be headed straight for New Orleans, but suddenly turned east, making landfall near Biloxi, MS on September 28th (about 68 miles east northeast of New Orleans). Georges produced sustained winds of over 100 mph at landfall, generating a storm surge of 8.9 ft at Point à la Hache, LA. Maximum storm surge along the Gulf Coast was 11 ft, in Pascagoula, MS. Hurricane Georges severely eroded the Chandeleur Islands in outer St. Bernard Parish. Despite forewarnings and evacuation orders 460 people were killed, all outside of Louisiana. Dozens of camps not protected by levees were destroyed along the south shore of Lake Pontchartrain. Hurricane Georges provided the last pre-Katrina test of the vulnerability of New Orleans levee protection system to hurricanes, and efforts resumed to improve the levee system along the canals that connect the city with Lake Pontchartrain.

4.5 Flooding of New Orleans Caused by Intense Rain Storms

As mentioned previously, the New Orleans area receives an average of about 52 cumulative inches of rainfall each year. In the winter of 1881 severe rainstorms caused flooding of the downtown area, up to 3 feet deep. Rain storms of severe intensity also caused significant flooding of New Orleans in 1927, 1978 and 1995.

The 1927 storm dumped 14 inches on Good Friday, overwhelming the Sewerage & Water Board's vaunted system of Wood pumps, at least temporarily. Uptown streets were flooded, with the Broadmoor and Mid-City areas inundated by 6 feet of water and 2 feet in the old French Quarter. This storm occurred simultaneously with the onset of the record high flows along the lower Mississippi River, which lasted almost six months.

On May 3, 1978 a line of rain squalls approaching New Orleans from the west became stalled over the city when it intersected a stationary front sitting over Lake Pontchartrain. The resulting storm dropped 10 inches of rain during the morning, with a peak sustained intensity of two inches per hour rain. The runoff exceeded the aggregate capacity of the city's pumps operated by the S&WB, causing extensive flooding of low lying areas that lasted about 24 hours.

A series of intense rain storms struck Louisiana, Mississippi, and Alabama in two consecutive sequences in March and April of 1980. The first storm occurred from March 26 to April 2nd, striking southeastern Louisiana and portions of Mississippi. The second storm sequence rolled through the same area from April 11 to April 13, affecting much of Mississippi, but especially intense in the area bounded by Baton Rouge and New Orleans to Mobile, Alabama. The 2-hour rainfall in Mobile on April 13 had a recurrence interval of 100 years. As a result of this rainfall, Mobile experienced the worst flash floods in the city's history. In New Orleans flood waters being pumped into the London Avenue Canal overtopped the eastern side of the Canal just south of Robert E. Lee Boulevard, where steel sheetpiles providing additional flood freeboard had recently been removed. This was the same portion of the northern London Avenue Canal which subsequently experienced incipient failure during Hurricane Katrina in 2005, and it moved two feet laterally (the area shown in Figure 4.24 - upper).

On the evening of May 8-9, 1995 a cold front approaching New Orleans from the west stalled after moving east of Baton Rouge. A nearly continuous chain of thunder storms befell the New Orleans area, dropping 4 to 12 inches of rain across New Orleans. The storm's intensity overwhelmed the S&WB's maximum pump capacity (47,000 cfs) and almost the entire city experienced severe flooding, including the Interstate highways. More severe storms struck the coast the following evening, but the rainfall was not as severe over New Orleans proper, though the two day totals reached a record 24.5 inches in Abita Springs, LA. The 1995 storm sequence had a duration of 40 hours and damaged 44,500 homes and businesses, causing \$3.1 billion in damages. This was the costliest single non-tropical weather related event to ever affect the United States.

4.6 New Orleans Drainage Canals

The drainage canals of New Orleans are a unique feature of the bowl-shaped city that are much older than most people realize. The city's first drainage canal was the Old Carondelet Canal originally excavated in 1794, by order of Spanish Governor Baron de Carondelet. It was dug by convicts and slaves and it was later enlarged to accommodate shallow draft navigation (row boats and keel boats) between the City and Lake Pontchartrain. Its name was later changed to the Basin Canal because it terminated at Basin Street, in the French Quarter. Its name was later changed to the Old Basin Canal. It was infilled in the 1920s, when it became Lafitte Avenue and railroad tracks were placed down the street's centerline. Figure 4.14 shows the systems of drainage ditches and canals established by 1829, leading to Bayou St. John.

The New Basin Canal was excavated by Irish immigrants in the early 1830s in the American Sector, but an outbreak of yellow fever killed 10,000 workers. The New Orleans City Railroad paralleled this canal in post Civil War era. The New Basin Canal was the first to cut through Metairie Ridge. The severing of Metairie Ridge was a double edged sword, as flood waters came up the Old Basin Canal and inundated the downtown area in 1871. The portion south of Metairie Ridge was filled in the 1930s; and the remainder in the 1950s, with the Pontchartrain Expressway replacing the old canal.

The six piece Topographic Map of New Orleans and Vicinity prepared by Charles F. Zimpel in 1833-34 suggest that portions of the Orleans Canal had been excavated and were proposed to be extended by that date to convey water from Bayou Metairie to Lake Pontchartrain (Lemmon, Magill, and Wiese, 2003). The Turnpike Road ran along the west side of this canal. In 1835 the New Orleans Drainage Company was given a 20-year charter by the city to drain the cypress swamps between the riverbank and Lake Pontchartrain. The company consulted State Engineer George T. Dunbar and evolved a scheme to drain the area using underground canals beneath prominent uptown streets which would collect water and convey it down the natural slope to the Clairborne Canal and then to the newly completed Orleans Canal (then called the Girod Canal) into Lake Pontchartrain. This ambitious scheme was derailed by the financial panic of 1837, though a system of ditches were completed which conveyed runoff from the French Quarter to the upper Orleans Canal, from which it had to be transferred to Bayou St. John using steam-powered pumps.

A review of historic maps (Figures 4.15 thru 4.17) suggests that the Upper Line Protection Levee or 17th St. Canal along the Orleans-Jefferson Parish boundary was excavated between 1854 and 1858 (shown as completed). The 17th Street Canal is not indicated on the 1853 Pontchartrain Harbor and Breakwater Map, although the Jefferson and Lake Pontchartrain Railroad is shown along the Orleans-Jefferson Parish boundary. The 1858 map shows the 17th Street canal just east of the railroad tracks and the new village of Bucktown, along the shore of Lake Pontchartrain adjacent to the mouth of the 17th St. Canal. The 1878 Hardee map (Figure 4.17) calls the 17th St. Canal the "Upper Line Protection Levee and Canal." 17th Street was renamed Palmetto Avenue in 1894. The early rail lines serving the docks on Lake Pontchartrain remained in operation for many years after the Civil War (Figure 4.16).

Disastrous outbreaks of yellow fever in the 1850s spurred new ideas to drain the cypress swamps. Between 1857-59 City Surveyor Louis H. Pilié developed a drainage plan using open drainage canals with four steam-powered paddle wheel stations to lift collected runoff into brick-lined channels throughout lower New Orleans, which was poorly drained because the Metairie-Gentilly Ridge presented a natural barrier between the downtown slope and Lake Pontchartrain (Figure 4.19). In 1858 the Louisiana Legislature divided the city into four "draining districts," providing a commission for each district and a method of assessment for the operation and maintenance of drainage facilities. These names of these were the New Orleans First and Second, Jefferson City, and Lafayette Draining Districts (Beauregard, 1859). In 1859 the legislature mandated issuance of 30-year bonds totaling \$350,000 for each of the four districts. This allowed a program of local taxation to fund the pumps and maintain the four lift stations, which were called "draining machines."

These steam-powered pumping machines were located at: the Dublin machine at the head of the New Canal (old 17th St.) at Dublin and 14th Streets; the Melpomene machine at the head of the Old Melpomene Canal (at Melpomene and Claiborne); the Bienville machine at the head of Bayou St. John (at Hagan and Bienville); and the London machine (just north of Gentilly and London Avenues). These facilities became a city trademark for many years thereafter. Shortly before the outbreak of the American Civil War in 1861, the legislature passed another bill that allowed any of the draining districts to make special assessments to make necessary repairs, based on the recommendations of their respective boards.

Figure 4.16 is a portion of the Map of New Orleans area completed under direction of Brigadier General Nathaniel P. Banks of the Union Army in February 1863, during the American Civil War. This map shows the position of the Jefferson and Lake Pontchartrain Railroad along the 17th St. Canal alignment, but not the canal itself. It also shows the New Basin Canal (a short distance east), the upper Orleans Canal, feeder canals emptying into Bayou St. John, and the Pontchartrain Railroad (near today's IHNC), which operated between 1831-1932, its northern terminus being named Port Pontchartrain.

The upper end of the London Avenue Canal appears to have been constructed in the 1860s, north of Bayou Gentilly. One of the afore-mentioned steam-powered draining machines was located near the intersection of London and Pleasure Street, which lifted water from the upper London Canal into the cypress swamp near what is now Dillard University, north of Gentilly Ridge. Based on a comparison of the 1873 Valery Sulakowski map and the 1878 Thomas Hardee maps, the lower London Avenue Canal appears to have been extended out to Lake Pontchartrain sometime between 1873-78.

In 1878 City Engineer and Surveyor Thomas S. Hardee compiled the most accurate map of the City to that date, after a yellow fever epidemic that year which killed 4% of New Orleans' population (which brought to City's accumulated death toll to Yellow Fever in excess of 100,000 people). The map sought to delineate improvements for the city's drainage system to enhance sanitation. It would take another two decades before a substantive drainage plan eventually evolved.

The New Orleans drainage dilemma can be appreciated from a review of the earliest cross section drawn through the city, reproduced in Figure 4.19. The Mississippi River's natural levees form the highest ground in New Orleans. The natural levee slopes northerly towards Lake Pontchartrain. This slope is interrupted by the Metairie-Gentilly Ridge, a geologically-recent distributary channel, lying between 3 and 6 feet above the adjacent swamp land.

The protection levee along Lake Pontchartrain (Figure 4.19) was erected after the 1893 hurricane, which generated a storm surge of up to 13 feet (described in Section 4.4). This protective structure was known as the "shoreline levee" and was 6 feet above the normal surface of Lake Pontchartrain. The creation of this structure was a double-edged sword: it served to keep rising water from Lake Pontchartrain out of the city, but also prevented gravity drainage from the city into the Lake, except through drainage canals, into which runoff must be pumped to gain sufficient elevation to flow by gravity into the Lake. Discharge could not be conveyed to Lake Pontchartrain during hurricane-induced storm surges. The gravity of this problem was not fully appreciated until the 1915 Grand Isle Hurricane.

4.7 City Adopts Aggressive Drainage System

The failure of the Hagan Avenue Canal levee in 1871 signaled the beginning of a political crisis, hastened by hurricane-induced surges on Lake Pontchartrain. The City sought to consider a better solution than it had heretofore employed in providing for reliable drainage to Lake Pontchartrain, and vice versa. New Orleans City Surveyor W.H. Bell warned of the potential dangers posed by the big outfall drainage canals. He told city officials to place pumping stations on the lakeshore, otherwise *“heavy storms would result in water backup within the canals, culminating in overflow into the city.”* This prophetic warning was ignored with catastrophic results during Hurricane Katrina.

A new attempt to construct an integrated drainage system was undertaken by the Mississippi and Mexican Gulf Ship Canal Company, which excavated many miles of canals in New Orleans between 1871-78, before going out of business. By 1878 the City assumed responsibility for maintenance of a 36-mile long system of drainage canals feeding into Lake Pontchartrain. The city’s old network of steam-powered paddle-wheel lift stations could only handle 1.5 inches of rainfall in 24 hours, which represented slightly more than a nominal 1-year recurrence frequency storm. This meant that the city began suffering flooding problems with increasing frequency because of insufficient runoff collection, conveyance, and pumping/discharge capacity.

The drainage problem was greatly exacerbated by a growing sewage treatment crisis. The City’s population grew from about 8,000 in 1800 to nearly 300,000 residents by 1900. The need for space enticed development into the low lying cypress swamps, which were being reclaimed by construction of shallow drainage ditches feeding into the newly completed system of drainage canals. In the 1880s houses began to appear on the old marsh and swamp areas below Broad Street. No one regulated the inflow to the drainage canals and there was an abject lack of a modern sewerage collection, conveyance, treatment, or outfall system. Residents on the high ground near the Mississippi River could install pipes that conveyed their effluent to the Mississippi River, but this was not a practical option for people living below Broad Street, which lay below the river level.

The drainage crisis grew throughout the 1880s. In 1890 the Orleans Levee Board offered \$2500 for the best drainage plan for the troubled city, but no suitable plans were submitted because of the paucity of reliable topographic data. In the wake of this disappointing result, newspaper editorials and civic leaders recognized the city could not continue growing without a substantive effort to handle drainage and sewage. After several more unsuccessful attempts to encourage someone credible to come forward with a plan, in February 1893 the City Council created a Drainage Advisory Board (DAB) and provided \$700,000 to gather the necessary topographic and hydrologic data, study the situation, and make recommendations on how the problems might be solved. The DAB sought to gather together the City’s best and brightest engineers from public, private, and academic ranks. Chief among this work was the preparation of an accurate topographic map of the city, prepared under the direction of City Engineer L. W. Brown (shown in Figure 3.22).

The first DAB’s findings were presented to the city in January 1895 (Advisory Board, 1895; Kelman, 1998). The Drainage Board recommended that the city create a modern system of drainage collection, conveyance, and discharge, which included street gutters, drop

inlets, buried storm drains beneath city streets, with gravity flow to the principal drainage canals leading to Lake Pontchartrain. At that juncture, the conveyance problems became unprecedented, insofar that the city would need to install a series of pump stations to convey collected runoff into Lakes Borgne and Pontchartrain. The projected cost of such a system would be enormous.

The following year (1896) the Louisiana legislature authorized the creation of the Drainage Commission of New Orleans, which began preparing a comprehensive drainage plan for the city, and, a corollary plan to fund such work. In 1897 the Drainage Commission began issuing contracts for new pumping stations, an electric power generation station, and the construction of additional feeder canals into the existing network of drainage canals.

In June 1899 voters passed a municipal bond referendum in a special election, which allowed a property tax of two mills per dollar to fund municipal waterworks, sewerage and drainage. With this revenue mechanism in place, the Sewerage & Water Board (S&WB) of New Orleans was shortly thereafter established (in 1899) by the State Legislature to furnish, construct, operate, and maintain a water treatment and distribution system and sanitary sewerage system. In 1900 the Drainage Commission began re-aligning and shifting the existing system of drainage canals, filling in a number of the cross-cutting canals and feeder canals which contained much stagnant water, which was encouraging the proliferation of mosquitoes and summertime yellow fever epidemics. In 1903 the S&WB was merged with the Drainage Commission to consolidate operations under one agency for more efficient operations. The drainage infrastructure at this time is shown in Figure 4.20.

The combined organization retained the name Sewerage & Water Board (S&WB), which it retains today. S&WB then set about the Herculean tasks at hand, which more or less continued at a feverish pace until the early 1930s, when the economic downturn caused by the Great Depression curtailed revenue. By 1905 the S&WB had completed 40 miles of drainage canals (in addition to the 36 they inherited), constructed six new electrically powered pumping stations and had a pumping capacity of 5,000 cfs, which represented about 44% of the original plan. At this time the S&WB provided drainage for 34.4 mi² of city area, all on the eastern side of the Mississippi River.

As the S&WB tackled the tough drainage problems plaguing lower New Orleans, rapid development of these low lying areas ensued, with the real estate values increasing dramatically, with many of the city's residents engaged in speculation, purchasing lots and then selling them as prices inflated. Because of this, many of the lots in lower New Orleans were developed in different eras instead of all at once, leading to the heterogeneity of architectural styles and ages that have made New Orleans neighborhoods famous. An unforeseen downside of the rapid pace of development was the increase in runoff which accompanied the emplacement of impervious surfaces, such as streets, roofs, sidewalks, and the like, which increased drainage problems, necessitating enlargement of pump capacity each decade.

By 1910 the S&WB system was rapidly being overwhelmed and something needed to be done to increase capacity. A. Baldwin Wood was a young Sewer & Water Board mechanical engineer who joined the Sewer & Water Board as assistant manager of drainage upon his graduation from Tulane University in 1899. Wood was a retiring and shy

personality who took on the various challenges facing the S&WB with unparalleled enthusiasm and imagination. Within a few years (at age 27 in 1906) Wood filed his first patent, for a 6-ft diameter centrifugal water pump that was the largest of its kind in the world. After this he invented an ingenious flap-gate that prevented backflow when the pumps were not in use.

In 1913 Wood made his greatest contribution to the continued growth of New Orleans when he introduced his novel design for the low-lift “Wood Screw Drainage Pump,” a 12-foot diameter screw pump that employed an enormous impellor powered by a 25 cycle per second (or Hertz, abbreviated as Hz) Alternating Current (AC) electrical motor. The motive power was highly efficient, using 20 feet diameter Allis Chalmers dynamos that spin up to 87 rpm. The low-lift screw pumps employ a siphon action to maximize hydraulic efficiency. This was followed in 1915 by Wood’s patented Trash Pump, capable of pumping record volumes of water as well as flotsam and trash without risk of shutting down the pumps (Junger, 1992). This latter feature was of particular value in maintaining pumping during storm events, which brought large volumes of organic debris into the drainage canals. In 1915 the City let a \$159,000 contract for thirteen patented Wood screw pumps, installing 11 of them in three pump stations by the end of the year, when the Grand Isle Hurricane struck the city, causing widespread flooding of the old back swamps, which already lay at sea level. By that time (1915) there were 70 miles of drainage canals in place.

By 1926 the New Orleans S&WB was serving an area of 47 mi² with a 560 mile long network of drainage canals and storm drains with a total pumping capacity of 13,000 cfs. This impressive infrastructure had been constructed over a period of 47 years at a cost of \$27.5 million (1879-1926). Up to this time (1926) most of the S&WB’s revenue had been generated by the special two-mill tax on all property and half of the surplus from the 1% debt tax. As the city grew and the S&WB’s jurisdictional area increased to other areas adjacent to the city, the tax structure saw a number of amendments. Today the S&WB is funded by a number of sources, including three, six, and nine-mil property taxes.

The integrated drainage network allowed the water table of the old cypress swamps to be dropped so that subterranean cellars and burials became possible, and deaths from malaria and typhoid dropped 10-fold between 1899-1925. The City’s last bout with summertime yellow fever was in 1905 (Campanella, 2002). During this same interim (1915-26), the port authority saw enormous growth with the development of a massive Army Supply Depot along the riverfront during the First World War (1917-18) and the long-anticipated completion of the Inner Harbor Navigation Canal (IHNC) between the river and Lake Pontchartrain in mid-1923.

In the mid-1920s Wood increased the capacity of his patented screw pump to 14 feet diameter, using the same powerful siphon action to lift water. This increased the capacity of each pump unit by almost 40%. His improved capacity screw pumps were eventually marketed across the world; in China, Egypt, India, and Holland. Wood retired from the S&WB in 1945 and died in May 1956.

4.7.1 Pre-Katrina Conditions and Maintenance by the S&WB

Today the S&WB is responsible for draining 95.3 mi² of New Orleans and neighboring Jefferson Parish, which receive an average annual rainfall of 52 inches per year. The general layout of the drainage system is presented in Figure 4.22. The pre-Katrina system was intended to handle an average annual discharge of 12.9 billion cubic feet of water that had to be collected and pumped into Lake Pontchartrain, Lake Borgne, and the Mississippi River. The City's 22 main pump stations and 10 underpass pump stations still use about 50 of A.B. Wood's old pumps, and their system can lift an aggregate total of 47,000 cfs of water under peak operating conditions (the State Department of Transportation maintains the pumps for the General DeGaulle underpass at the Mississippi River Bridge ramps and on the East Bank at the Pontchartrain Expressway at the Southern Railway tracks and Metairie Cemeteries). A typical pump station (Pump Station No. 6) can lift 9,600 cfs using its old Wood pumps. New Orleans also employs vertical pumps with impellers to lift water from subterranean (below street) storm drains to the drainage canals, which outfall in Lake Pontchartrain. The S&WB maintains 90 miles of covered drainage canals, 82 miles of open channel canals, and several thousand of miles of storm sewer lines feeding into their system.

The S&WB maintains that their agency installed two sets of piezometers along the canal in the early 1980s, but that these revealed little correlation between transient flow levels in the canals and the adjacent piezometers. They took this result to mean that the canal floored in materials of relatively low permeability. In 1988 the S&WB received a permit from the Corps of Engineers to deepen and widen the 17th Street Canal, based on the "positive" indicators garnered from the piezometers that had been installed a few years previous. The Corps warned that dredging might weaken the stability of the canal, but a system of monitoring pore water (groundwater) pressures adjacent to the canal was not undertaken and the canal was substantially enlarged using a track-mounted excavator.

Although the S&WB system is highly efficient from an energy expenditure perspective, the 25 Hz AC electrical power requires the board to produce its own electricity, in lieu of purchasing 60 Hz AC off the national electrical power grid. As a consequence, approximately 60% of the S&WB's electrical power has to be generated locally, at their own 20 MW generator stations (Snow, 1992). Unfortunately, all of these generating stations are located below mean Gulf level and subject to shut-down by flooding.

4.7.2 Damage to S&WB Facilities and Capabilities Caused by Hurricanes Katrina and Rita

During Hurricane Katrina the following pump stations were incapacitated and closed due to flooding: Pump Station #1 (2501 S. Broad Street), #3 (2252 N. Broad St.), #4 (5700 Warrington Dr.), #6 (345 Orpheum), #7 (5741 Orleans Ave.), #10 (9600 Haynes Blvd.), #14 (12200 Haynes Blvd.), #15 (Intercoastal Waterway), #16 (7200 Wales St.), and #19 (4500 Florida Ave.). These pump stations were gradually brought back online and were all at least partially operational within six months. 100% pumping capacity had not been restored to the S&WB system by the time of this writing (May 1, 2006). Drainage for Jefferson Parish, west of the city, remained online in wake of Hurricanes Katrina and Rita. This failure of the

S&WB drainage system was without historic precedent, and pointed to fundamental flaws in the drainage system, with respect to operational redundancy.

During Hurricanes Katrina and Rita the Eastbank Sewer Treatment Plant was also closed (and has not reopened as of June 1, 2006). City residents were immediately advised to boil water before using it by the city's Department of Health and Hospitals immediately following flooding of the city. This restriction was lifted for the neighborhoods west of the IHNC on October 6, 2005 and for the New Orleans East, Southshore and Ventian Isles areas on December 8, 2005. Water quality had not been restored to The Lower Ninth Ward in Zip Code 70117 by the time of this writing (June 1, 2006).

4.7.3 Reclamation of the Mid-City Lowlands (early 1900s)

The Mid-City area occupies a natural basin that formed between the levees of the Mississippi River and Metairie Ridge. The City's original network of pie-shaped property boundaries and streets converged on this area from their points of origin perpendicular to the broad crescent-shaped bend of the Mississippi River upstream of the French Quarter, from which the city derives its motto "the Crescent City." The area was a closed depression (Figure 4.18), which had to fill up with water to drain into Bayou St. John, thence three miles into Lake Pontchartrain. A series of feeder canals were excavated to convey drainage into Bayou St. John and the New Basin Canal after the Civil War. But stagnant water occupied these feeder ditches, promoting the existence of mosquitoes and yellow fever outbreaks, which were recognized to favor poorly drained areas decades before the scientific connection between the two was established (beginning around 1905).

In the early 1900s it was decided to begin filling the lowest areas of the Mid-City area to provide better drainage and accommodate growth into this area, which had been subject to frequent flooding. Sand from Metairie Ridge and from dredging of nearby canals was used to provide the fill material and the feeder canals in this area were filled in and replaced with buried storm drain pipes beneath the streets (discussed in Section 4.7).

4.7.4 1915 Flood Triggers Heightening of Drainage Canal Levees

On September 29, 1915 The Grand Isle Hurricane lifted the water level in Lake Pontchartrain to 13 feet above mean gulf level. The Lake Pontchartrain shoreline levee and many of the drainage canals were overtopped and much of the lower city flooded, killing 275 people. The City's new pump system was overwhelmed when the power generating stations for the new Wood screw pumps were flooded. After the 1915 flood, Sewerage and Water Board General Superintendent George Earl ordered the levees along the drainage canals to be raised approximately three feet, while the Pontchartrain shoreline levee was also raised. It is not known if this work was carried out by the S&WB or the Orleans Levee District.

4.7.5 The Lakefront Improvement Project (1926-34)

The southern shore of Lake Pontchartrain supported a number of small commercial wharves and fishing camps during the late 19th Century, including Milneburg, Spanish Fort, and West End. Shanties and structures along the shore were founded on wood pilings. The

old Lake Pontchartrain shoreline levee had been constructed along the south shore to protect New Orleans from flood surges off the lake around 1893. This levee was overtopped by the storm surge on Lake Pontchartrain during the Grand Isle Hurricane in 1915 (described in Section 4.4). This levee was difficult to maintain because the shoreline was actively receding southward, towards New Orleans (Figure 3.16). In 1921 the Orleans Levee Board were granted increased powers by the state legislature to reinforce the Pontchartrain shoreline. In 1924 the board's chief engineer, Colonel Marcel Garsaud, embarked on developing an ambitious plan to construct a permanent seawall along Pontchartrain's south shore and reclaim several square miles of land by filling the gap between the new seawall and the eroding shoreline.

In 1926 the levee board began construction of a temporary wooden bulkhead wall constructed one-half mile north of the existing shoreline, within Lake Pontchartrain. This temporary structure extended two feet above mean gulf level (MGL). The nearshore area between this bulkhead wall was initially backfilled to an elevation of +2 feet above MGL, creating 1,800 acres of "made ground." The fill material was sand taken from the floor of Lake Pontchartrain, placed using hydraulic dredges. The wooden bulkhead was then raised another two feet and hydraulic fill placed behind it to a level of +4 ft. This process was repeated yet again, creating a fill platform 4 to 6 feet above MGL and up to 10 ft higher than the old cypress swamps that subsequently became the Lakeview and Gentilly neighborhoods (even higher than the Metairie-Gentilly Ridge). The reclamation plan envisioned the construction of a permanent stepped concrete seawall along the new shoreline, replacing the wooden bulkhead wall, and construction of this permanent barrier began in 1930.

To offset the hefty price tag of \$27 million for this work, the levee board secured special legislation (in 1928) creating the Lakefront Improvement Project, which allowed them sweeping powers to reclaim land along the Pontchartrain shoreline. In 1931-32 another sizable fill was placed along Lake Pontchartrain behind another concrete seawall to create an additional 300-acre fill for a municipal airport. This was christened Shusan (now Lakefront) Airport, which has a 6,900 ft runway, used as a flight training facility during World War II.

When the lakefront improvement project was completed in 1934, a public debate erupted as to how best utilize the reclaimed land. A battle soon developed between private development, public access to the shoreline, and those forces promoting its adoption as open space parkland. A compromise plan was eventually adopted which allowed public access for recreation along with residential and public facility development (University of New Orleans). The new acreage was sold to developers to help the levee board pay off the construction bonds, and the Lakeshore, Lake Vista, Lake Terrace, and Lake Oaks neighborhoods were developed between 1939-1960.

After the Second World War the Lakeview, City Park, Fillmore, Gentilly, and Pontchartrain Park areas behind the lakefront emerged as desirable bedroom communities with yacht harbors, parks, and pleasant summer breezes. This area experienced unprecedented growth, between 1945-75, adding about 100,000 residents to the City.

4.7.6 Second Generation of Heightening Drainage Canal Levee Embankments (1947)

The hurricane of September 1947 caused storm surges of up to 10 ft above MGL along the shores of Lakes Borgne and 5.5 ft along the south shore of Lake Pontchartrain which overwhelmed levees in the Inner Harbor Navigation Canal (IHNC) and the old drainage canals, within a mile of their respective mouths. After several of these drainage canal levees were overtopped in 1947, the state's congressional delegation asked the federal government to assist in protecting the city (culminating in the Lake Pontchartrain and Vicinity Hurricane Protection Plan passed by Congress in 1955). The Orleans Levee Board spent \$800,000 to raise its levees, including both sides of their drainage canals (with the exception of 17th Street, the west side of which is owned by the Jefferson Levee Board). Sheet piles were also reportedly used in by the port authority in the inner harbor area. We have not been able to determine how much additional freeboard was added by filling and/or sheet pile extensions in 1947-48.

4.7.7 Federal Involvement with the City Drainage Canals (1955 – present)

Federal involvement in the city's drainage canals began in 1955 with approval of the Lake Pontchartrain and Vicinity Hurricane Protection Project by Congress. The Corps studied the problems posed by the drainage canals, which had settled as much as 10 feet since their initial construction in the mid-19th Century. This settlement had necessitated two generations of heightening following hurricane-induced overtopping in 1915 and 1947. Each of these upgrades likely added something close to three additional feet of embankment height to keep water trained within the drainage canals and provide sufficient freeboard to prevent storm surges emanating from Lake Pontchartrain from overtopping the canal levees. The maximum design capacity of the three principal drainage canals (17th Street, Orleans, and London Avenue) was about 10,000 cfs, but this figure was being reduced by settlement and sedimentation problems.

The Corps had several non-federal partners in the venture: the Orleans and Jefferson Parish Levee Boards, and the Sewerage & Water Board of New Orleans. The levee districts maintained the canals and the S&WB maintained the pump stations and controlled the discharge in the drainage canals. If the S&WB pumped at maximum capacity, the increased flow could accelerate erosion of the unlined canals, which floor in extremely soft soils. If they didn't pump much water, then the canals could fill up with sediment, and thereby experience diminished carrying capacity. By the time the Corps got involved, a dense network of single family residences abutted the drainage canals along their entire courses (the canals are 2-1/2 to 3-1/2 miles long). The encroachment of these homes adjacent to the canal embankments circumvented any possibility of using conventional methods to heighten the levees, which is usually accomplished by adding compacted earth on the land-side of the levees (Figure 4.23, which would require the condemnation and removal of hundreds of residences, which would be costly and time-consuming (not to mention unprecedented).

In 1960 the Corps of Engineers New Orleans District office issued its initial report detailing their plan for remedying the ongoing problems with the slowly sinking drainage canals. The Corps plan opted to solve the drainage canal freeboard problem by installing tidal gates and pumps at the drainage canal outfalls along Lake Ponchartrain. This obviated the need for condemning all the homes built along the canal levees. The Corps soon found itself

embroiled in a clash of cultures and goals with the levee districts, the S&WB, and the local citizenry, who flatly opposed the Corps' proposal. The S&WB and local residents feared that the tidal gates would malfunction, inhibiting outflow of pumped storm water, which would, in turn, allegedly cause flooding.

The following year (1961) the Corps of Engineers unveiled a more grandiose plan to provide hurricane flood protection for New Orleans by constructing large flow barriers at the passes (The Rigolets) leading into Lake Pontchartrain, to prevent storm surges from reaching the lake. This scheme was expensive, and never garnered sufficient political support to gain appropriations (it was also proposed in the era before environmental assessments were required).

The issue of how to address improvement of the drainage canals dragged on for another 17 years. Between 1960-77 what few lots remained in lower New Orleans were rapidly built out, and most of the post-1970 development in New Orleans focused on the areas east of the IHNC, in Jefferson Parish (west of New Orleans), and across the Mississippi River (Algiers, etc). In 1977 the U.S. Circuit Court of Appeals ruled against the Corps of Engineers plans for tidal gates at the mouths of the drainage canals because the Corps failed to examine the impacts of alternative schemes. From this juncture, the Corps focus shifted to heightening the drainage canal levees using concrete walls (Figure 4.24-lower), which was what the opposing groups desired. These walls were to be designed to withstand a Category 3 storm surge with 12 ft tides and 130 mph winds.

Construction began in 1993, but the wrong benchmark datums were selected for the contract drawings, so some of these walls were constructed almost two feet lower than intended (IPET, 2006). Although the concrete flood walls were completed by 1999, concrete skirt walls on several of the bridges crossing the drainage canals had not yet been completed when Hurricane Katrina struck on August 29, 2005. So, the drainage canal system was not fully "tight," but it was generally believed that it could survive a Category 3 storm surge by surviving 6 to 8 hours of overtopping. The design storm surge values used by the Corps of Engineers are reproduced in Figure 4.25.

Records for the drainage canals in New Orleans indicate that between 1932-2005, water levels in these canals exceeded a flow stage of greater than +4 ft MSL on at least 29 occasions; +5 feet was exceeded 13 times (including during Hurricanes Betsy in 1965 and Camille in 1969; +6 ft was exceeded only three times (including during Hurricanes Juan in 1985 and Isadore in 2002); and exceeded +7 feet for the first and only time on August 29, 2005, during Hurricane Katrina.

4.7.8 Hurricane Katrina strikes New Orleans – August 2005

A complex network of levees protected the City of New Orleans from flooding (Figure 4.26). New flood walls were constructed in the 1990s on the crowns of drainage canals and the Inner Harbor Navigation Canal to accommodate functionality during high storm surges. The walls in the lower Lakeview and Gentilly Districts topped out at +14 ft above MGL.

This system of flood walls quickly failed on the morning of August 29, 2005, when water levels rose more than 7 feet above MSL, higher than ever previously recorded in the

drainage canals since 1932 (cited in previous section). Prior to Hurricane Katrina, the drainage canals feeding into Lake Pontchartrain never exceeded a flow height of between 6 and 7 feet above MGL. Many of the recording tidal gages failed during Hurricane Katrina. The incomplete record of the gage located closest to the 17th Street Canal failure is reproduced in Figure 4.27. This record shows several interesting trends. The first is the increase in diurnal high tide level each day after August 22nd. The second is a dramatic departure from the normal tidal cycle beginning the day before Hurricane Katrina made landfall, around 5 PM on August 28th. The third interesting aspect is the sharp increase in surge level on the morning of August 29th, which is much steeper than the assumed design storm surge for Lake Pontchartrain shown on the lowest curve in Figure 4.25.

4.8 Commercial Navigation Corridors

4.8.1 Inner Harbor Navigation Canal/Industrial Canal

Ever since the founding of the city by the French in 1718, the concept of a navigation channel between the Mississippi River and Lake Pontchartrain had been proposed, which would allow intercoastal commerce to connect with river and seaborne commerce traveling up and down the Mississippi River. The Port Authority of New Orleans was established in 1896 as an agency of the State of Louisiana. The port engineers recognized that the problem with establishing a water borne link was the fluctuating flow of the river, which raised and lowered 20 feet, depending on flood stage. The river was also 10 to 26 feet higher than the normal level of Lake Pontchartrain, so some impressive locks would be needed to control the flow between the river and the lake.

The idea never progressed too far until construction of the Panama Canal between 1906-14, which heralded advances in excavation and grading technology that allowed widespread programs of public works, drainage, and flood control in the succeeding half century. In July 1914 New Orleans received authorization from the state legislature to locate and construct a deep water canal between the Mississippi River and Lake Pontchartrain, which was supposed to boost the capacity of the port by as much as 100%. America's entry into the First World War triggered the rapid expansion of ship building facilities and construction of an enormous Army Supply Depot along the river front. While the war was still raging, a committee was formed early in 1918 to examine the feasibility of a connecting canal, using the most modern technology. Their initial report was released in May 1918 and it surprised everyone by envisioning a much larger project than most supposed, with the creation of ship building facilities within a protected, fixed-level harbor, increasing the available wharf space by almost 60%. The canal would be 5.3 miles long and up to 1,600 ft wide, located just downstream of the Army's new riverfront Supply Center (about 2 miles downriver and parallel to Elysian Fields Avenue). A key aspect of its location was the 1911 donation of a pie-shaped tract of land owned by the Ursuline Nuns which covered about half of the proposed route, contiguous with the Mississippi River.

The Port Authority's Dock Board retained the services of the George W. Goethals Company as consulting engineers, borrowing upon General Goethals renown as chief engineer of the Panama Canal project a few years earlier. The local firm of J. F. Coleman Engineering Co. performed most of the actual detailed design work, as well as assisting the

Port Authority in construction management. Construction commenced on June 6, 1918. The superior elevation of the Mississippi River dictated that excavation would necessarily proceed from the lake side towards the river, and the massive locks, the project's kingpin structure, would be placed at the river end of the canal.

Excavation work initiated with the construction of parallel dikes on either side of the proposed canal, from which hydraulic fill could be loosed through sluice pipes. Hydraulic excavation was used wherever possible to excavate the channel, when the materials were easily loosed (e.g. low cohesion materials, such as gravel, sand, organic ooze and swamp muck). When more resistant clay was encountered large front tower cableway dragline excavators or conventional dragline excavators (Figure 4.29) were employed to scoop out the clay and drag it up onto the dikes, which were gradually built up to become permanent protective levees. The draglines employed 3.5 cubic yard buckets and could handle about 150 cubic yards per hour. From the onset, contractors battled problems with slope stability, as the soft oozy soils constantly slid back into the excavation (Campanella, 2002). Buried cypress stumps slowed progress by jamming suction dredges and stalling dragline buckets.

During construction the Port Authority decided to increase the size of the channel to a minimum depth of 30 feet at low water, with a minimum bottom width of 150 feet and a minimum channel width of 300 feet, roughly double the original design. Abreast of the new wharves the bottom width was increased to 300 feet, with a minimum canal width of 500 feet near piers and slips, and 600 feet adjacent to quays (Dabney, 1921). The canal excavation was completed in just 15 months, in September 1919. Everyone's attention then turned to the lock structure, located 2,000 ft from the Mississippi River, at the south end of the canal. The normal flow level of the river was 10 ft above that of Lake Pontchartrain, so cofferdams had to be constructed on either end of the locks to allow safe access and dewatering of the exposed foundations. The lock is 640 ft long and 74 ft wide. The footing excavations were 50 feet deep, where timber piles were pounded into the underlying sands. The lock structure was finally completed on January 29, 1923, and dedication ceremonies for the entire Inner Harbor Navigation Canal (IHNC) were convened on May 5th, 1923. The residents of New Orleans often refer to the IHNC as the "Industrial Canal."

Almost immediately upon completion, the Port Authority set about developing piers, docks, and quays to increase cargo handling. Their first large structure was the Galvez Street Wharf, which was 250 ft wide and 2,400 ft long, costing \$1.8 million (1923 dollars), completed in 1924. It was constructed of reinforced concrete and fitted with tracks for a local Beltline railroad. The Port Authority also made available adjacent lands for use by industries, but it took many years until the envisioned development occurred. The IHNC benefited from the completion of the Intracoastal Waterway in the mid 1930s, as a cargo handling and provisioning stop. This was an unforeseen benefit, serving smaller vessels, which provided an economical means of transport prior to the establishment of the Interstate Highway network in the 1960s.

The massive Florida Avenue Wharf was added during World War II while the Gentilly Road section of the canal witnessed the sprawling expansion of shipbuilding facilities operated by Andrew Jackson Higgins, who pioneered the development of wooden PT boats and landing craft crucial to the war effort. Much of the area flanking the west side of the IHNC was built out during World War II (Figure 4.29). The eastern side was developed

much later, after the Korean War (1950-53) and completion of the MRGO channel in 1964 (Figure 4.28). The immense France Road and Jordan Road Container Terminals (Berths 5 and 6) near the head of the MRGO channel were completed in the 1980s and 90s. The narrow width of the 1923 lock (74 feet) has restricted the passage of commerce, in particular, river barges, which often wait up to 36 hours to pass through.

4.8.2 Flooding problems around the IHNC

During the 1947 hurricane (Figure 4.11) a back protection levee adjacent to the IHNC was overtopped at Tennessee Street, spilling 10 feet of water into the East Side of New Orleans. Fortunately, the levee did not collapse, the area was undeveloped, and the flooding was quickly cleaned up. There was also quite a bit of flooding in the Metairie and Jefferson Parish areas, also attributable to temporary overtopping. There was a flood inundation map published in the *New Orleans Times-Picayune*.

Both sides of the IHNC experienced breaks and overtopping during Hurricane Betsy in September 1965. 6,560 homes and 40 businesses were flooded in water up to 7 ft deep on the west side of the IHNC. The east side of the IHNC also failed, flooding the west end of St. Bernard's Parish. A map of the flood inundation of New Orleans caused by Hurricane Betsy in September 1965 is shown in Figure 4.12. The Corps' report on Hurricane Betsy (USACE, 1965) states that both internal levee failures and overtopping occurred along the Inner Harbor Navigation Canal, on both the west and east sides. No details about the mechanisms of failure were described, however.

The IHNC was heightened using steel sheetpiles and concrete I-walls in the 1980s and 90s. On August 29, 2005 during Hurricane Katrina both sides of the IHNC were overtopped by the storm surge converging on the IHNC from Lakes Borgne and Pontchartrain. Sustained overtopping flow undermined the landside toe of the I-walls, in places gouging down as much as 5+ feet below the crest of the earthen levee. In addition, there was ample physical evidence of underseepage at both the eastern IHNC breaches, in the form of linear sand boils.

4.8.3 Gulf Intercoastal Waterway (GIWW)

The Intracoastal Waterway (GIWW) was originally conceived in 1808, but was not authorized by Congress until 1919. The GIWW was excavated by dredge in the late 1930s to a channel size measuring 9 ft deep by 100 feet wide, and completed between New Orleans and Corpus Christi, Texas by mid-1942. This was enlarged to 12 feet deep by 125 ft wide channel and officially completed in June 1949. The GIWW forms a protected shipping lane between Port Isabel, Texas (the Mexican border) and Apalachee Bay, Florida. The first 15% of the Mississippi River-Gulf Outlet Channel follows the GIWW, which then diverges northeastward, about five miles east of the Inner Harbor. The GIWW then runs east, towards The Rigolets and onto the Mississippi coast.

4.8.4 Mississippi River Gulf Outlet

When the IHNC was completed in 1923 the Port Authority announced that it intended to lobby the federal government to construct a Mississippi River Gulf Outlet (MRGO)

channel connecting to the IHNC, to increase shipping capacity (Dabney, 1921). The idea didn't surface appreciably until 1943, during the Second World War, when thousands of amphibious assault craft and shallow draft vessels were being fabricated along the nation's inland waterways. The Corps of Engineers felt that a tidewater canal serving New Orleans and the nation's interior waterways would be able to compete with the Panama Canal for east-west shipping, crucial to the war effort (most industrial goods were manufactured in the eastern United States, which was being shipped to the Pacific via the Panama Canal). Competing priorities placed the project in limbo until the late 1940s, when it was resurrected. In the early 1950s the project was repeatedly voted down in Congress, because of competition with the St. Lawrence Seaway project between Canada and the U.S (approved in 1954).

After passage of the competing seaway, the Mississippi River Gulf Outlet (MRGO) project was authorized by Congress in March 1956. Kolb and Van Lopik (1958) of the Corps of Engineers prepared a geology report on the MRGO alignment in 1957-58. This study showed that the upper 2 to 5 feet was mainly fibrous peat, although highly organic marsh deposits extend to depths of between 5 and 16 feet. These highly compressible materials are underlain by interdistributary and intratidal complex silts and clays over much of the proposed alignment (Figure 4.31). They graded these materials as soft marsh (500 to 900% water content), firm marsh (100 to 500% water content), and swamp substrate (highly organic peat with 600 to 800% water content). They noted that the soft marsh and swamp substrate materials would be unable to provide competent foundations for the protective levees bordering the channel, and these same materials would be unsuitable for use in such embankments.

During the first phase of dredging in 1958-59, 20 million cubic yards (mcy) of material was excavated between the IHNC and Paris Road (now I-510), essentially widening the GIWW. In 1959-60 contractors excavated a "pilot channel" between the GIWW and Breton Sound, excavating and placing 27 mcy of material. In the third and fourth phases completed between 1960-65, 225 mcy were excavated between Paris Road and Breton Sound. Dredge spoils were placed in a strip of land 4000 ft wide along a corridor paralleling the southwest side of the MRGO channel in St. Bernard Parish. The dredge soils from the initial excavations (1958-59) were placed on the land which now underlies the Jourdan Road Container Terminal, near the intersection of the MRGO and IHNC.

The MRGO channel was excavated as 500-foot minimum width channel with a minimum (low tide) depth of 36 feet (excavated to -38 feet; accepted at -36 ft). The route of the MRGO channel crosses 45 miles of delta marshland in Orleans and St. Bernard Parishes, with another 30 miles of open (dredged) channel across Breton Sound. This offshore section is slightly larger. Its 75 mile path is 37 miles shorter than that of the deep water navigation channel connecting New Orleans to the Gulf of Mexico via Southwest Pass. The project was finalized in 1968.

The flanking levees have experienced significant settlement since the project's completion, due to consolidation of prodelta clays underlying the flanking levee embankments, as well as plastic sagging due to low strength and creep properties of underlying organic material. The amount of settlement varies between 1.5 and 8 feet, depending on location. Many estimates have been offered regarding the tectonic rate of subsidence of the Mississippi Delta; from 0.4 ft/century (Saucier, 1963) to as much as 1.3

ft/century (Watson, 1982). The Corps of Engineers authorized two sequences of levee heightening to keep pace with ongoing settlement, but the third was delayed by funding problems and had not been emplaced when Hurricane Katrina struck in 2005.

Since its completion, the seaway has eroded to a width of 2000 ft in places (Coastal Environments, 1984), due in large part to ship wakes in the relatively confined channel. In addition, siltation necessitates ongoing dredging, which cost the Corps of Engineers about \$16 million per year. Salt water intrusion along the channel has impacted adjacent marshes, although significant quantities of salt water have not been conveyed inland during hurricanes, because the channel's width is relatively insignificant when compared to adjoining bodies of water, such as Breton Sound and Lake Borgne.

During Hurricane Katrina the levees fronting the MRGO channel were overtopped by the near-record storm surge that came from the east off of Lake Borgne. The overtopping caused by the severe storm surge quickly eroded the MRGO frontage levees in those reaches where the levees were comprised of materials with little or no cohesion and high organic content. In long stretches the entire levee was washed away down to its original marsh foundations without a trace (Figure 4.32).

4.9 Influence of Elevation Datums on New Orleans Flood Protection System

4.9.1 Introduction

Persistent subsidence of the Gulf Coast/Mississippi River Delta region has led to a complex relationship between the various geodetic datums used during historic surveys of the area. The underconsolidated and organic rich sediments of the Mississippi Delta are continually subsiding due to their compressible nature, the biochemical oxidation of the entrained organics, and all the other factors described in Section 3.7. Tectonic activity along active normal faults is also contributing to subsidence of nearly the entire Gulf Coast region. Rates of subsidence are highly variable throughout the region, resulting in a complex relationship between different geodetic datums at benchmarks in the New Orleans area. Subsidence combined with a slow rise in sea level (about 1 ft per century) has caused much of the Gulf Coast Region surrounding New Orleans to drop ten or more feet relative to sea level in historic times, both of which have made the city more vulnerable to tropical storms.

It is important to accurately determine elevations in relation to sea-level in order to design and construct flood protection systems in areas vulnerable to tropical storms. Unfortunately, outdated terrestrial datums were referenced when constructing many of the floodwalls protecting New Orleans. Variations of the NGVD29 datum were used, which is based on terrestrial reference points, not sea level. The use of the outdated datums also neglected subsidence and sea level rise, resulting in a lesser protection height than intended in the floodwall designs. The subsidence of the region has made the correlation of datums a complex task. No single conversion factor may be used when converting between two datums.

4.9.2 17th St. Outfall Canal

Between 1952 and 2005, there has been a 2.345 foot decrease in the elevation of the benchmark ALCO at the mouth of the 17th St. Outfall Canal due to subsidence and adjustment of datums. In 1952, the benchmark elevation was 8.235' while it had decreased to 5.89' by 2005 (post Katrina) according to the NGVD29 (1952) and LMSL (1983-1992) datums, respectively.

When the concrete I-walls were placed atop the 17th St. Outfall Canal Levees during the 1990's, their tops were to extend to an elevation of 14.0 feet according to the NGVD datum. Contract reports do not specify which NGVD epoch was to be used in design and construction. It is possible that NGVD29 (09 Apr 1965) was used. In addition, NGVD is a terrestrial datum and is not directly referenced to sea level as is LMSL. The top of the 17th St. Outfall Canal Floodwall is presently between 1.3 and 1.9 below the design level of 14.0 feet according to LMSL (1983-1992). This is likely due to the use of an outdated datum (1.6 feet of difference) and settlement of the levee embankments and floodwalls (0.3 feet).

4.9.3 London Ave. Outfall Canal

The floodwalls bordering the London Ave. Outfall Canal were also designed and built during the 1990's. According to contract documents, the NGVD29 (09 Apr 1965) datum was used. The use of an outdated, terrestrial datum in conjunction with settlement has resulted in the floodwall heights being 1.6-1.8 feet below their intended heights of 14.4 feet (LMSL (1983-1992)).

4.9.4 Orleans Outfall Canal

The NGVD29 (01 Sep 1982) datum was referenced during the design and construction of the Orleans Outfall Canal floodwalls in the 1990's. Presently, the floodwalls surrounding this canal are up to 0.8 feet lower than called for than the 14.0-14.9 foot elevation called for in the designs (according to LMSL (1983-1992)).

4.9.5 Inner Harbor Navigation Canal – East Levee

Floodwalls were placed atop the Inner Harbor Navigation Canal's East Levee in 1970. The walls were to extend to 15.0 feet (MSL) according to the 1969 contract documents. MSL was tied to an earlier terrestrial datum and the exact correlation to modern adjustments has yet to be determined. Floodwalls presently reach heights between 12.3 and 13.2 feet according to the LMSL (1983-2001) datum.

4.9.6 Inability to Apply Universal Corrections for Elevation Datums

Although subsidence has played a role in the differences between designed and actual floodwall heights, most of the variance appears to have been caused by datum abnormalities. It is standard engineering practice to use an NGVD datum to determine sea level. The use of NGVD is not cause for concern in portions of the country away from coastlines but becomes troublesome in areas at or just above sea level.

Due to the highly variable rates of subsidence throughout the region, a common conversion factor cannot be used to adjust between datums, even over a short distance. The complex relationships between the various geodetic datums in the New Orleans Region are not discussed in great detail in this report. A more thorough discussion of this subject is presented in Chapter III of IPET's Second Interim Report (IPET, April 2006).

4.10 Names of New Orleans Neighborhoods

Figure 4.34 presents the official neighborhood names recognized by the City of New Orleans. Local residents also use local ward and district numbers, and parish names to describe an area. A common example would be the Lakeview and Gentilly areas, which are used in a general sense to describe the former Cypress swamplands that now are among the City's lowest lying areas. The "Lakeview district" more or less encompasses Lakewood West End, Lakewood, Lakeview, Navarre, and City Park neighborhoods. The "Gentilly district" more or less includes the Fillmore, St. Anthony, Dillard, Milneburg, Gentilly Terrace, Pontchartrain Park and Gentilly Woods neighborhoods.

4.11 References

- Advisory Board [New Orleans]. (1895). Report of the Drainage of the City of New Orleans by the Advisory Board, Appointed by Ordinance No. 8327, Adopted by the City Council [of New Orleans], November 24, 1893.
- Barry, J. M. (1997). *Rising Tide: The Great Mississippi Flood of 1927 and How It Changed America*, Simon & Schuster, New York.
- Beauregard, P.G.T. (1859). *Report on Proposed System of Drainage for First Draining District, New Orleans*. Chief Engineer's Office, U.S. Corps of Engineers, New Orleans LA.
- Campanella, R. (2002). *Time and Place in New Orleans: Past Geographies in the Present Day*, Pelican Publishing.
- Chatry, F.M. (1961). "Flood Distribution Problems below Old River." *Transactions of the American Society of Civil Engineers*, 126(1), 106-119.
- Cline, I. M. (1926). *Tropical Cyclones, Comprising an exhaustive study of features observed and recorded in sixteen tropical cyclones which have moved in on gulf and south Atlantic coasts during the twenty-five years, 1900-1924 inclusive*, Macmillan Co., New York.
- Coastal Environments, Inc. (1984). The Mississippi River Gulf Outlet: A Study of Bank Stabilization. Consultant's report to St. Bernard Parish Police Jury, U.S. Department of Commerce National Oceanic and Atmospheric Administration, and State of Louisiana Department of Natural Resources. Baton Rouge. 127 p.
- Dabney, T.E. (1921). The Industrial Canal and Inner Harbor of New Orleans: History, Description, and Economic Aspects of Giant Facility Created to Encourage Industrial

- Expansion and Develop Commerce. Board of Commissioners of the Port of New Orleans.
- Denny, M. (2002). Surveying Little Egypt. Point of Beginning. Vol. 27:8 (April 30, 2002).
- Elliott, D.O. (1932). The Improvement of the Lower Mississippi River for Flood Control and Navigation. Mississippi River Commission, St. Louis.
- Fisk, H. N. (1952). *Geological Investigation of the Atchafalaya Basin and the Problem of Mississippi River Diversion*, Vol. 1, U.S. Corps of Engineers Waterways Experiment Station, Vicksburg LA.
- Hewson, W. (1870). Principles and Practice of Embanking Lands from River-Floods, as applied to "Levees" of the Mississippi, 2nd Ed., D. Van Nostrand, New York.
- Interagency Performance Evaluation Task Force (IPET). (2006). Performance Evaluation Status and Interim Results, Report 2 of a Series, Performance Evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System. Final Draft (subject to revision), 10 March 2006. 322 p.
- Jadwin, E. (1928). "The Plan for Flood Control of the Mississippi River in Its Alluvial Valley." *Great Inland Water-Way Projects in the United States*, R.A. Yound (Ed.), Philadelphia, American Academy of Political and Social Science, CXXXV (January), 35-44.
- Junger, S. (1992). "The Pumps of New Orleans." *American Heritage Invention & Technology*, 8(2), 42-48.
- Kelman, A. (1998). *A River and Its City: Critical Episodes in the Environmental History of New Orleans*, Ph.D. dissertation. Brown University, Providence RI.
- Kesel, R.H. (2003) "Human modification of the sediment regime of the lower Mississippi River flood plain." *Geomorphology*, 56, 325-334.
- Kolb, C. R. (1976). "Geologic Control of Sand Boils Along Mississippi River Levees." *Geomorphology and Engineering*, D.R. Coates (Ed.), Dowden, Hutchinson & Ross, Halsted Press, 99-113.
- Kolb, C.R., and Saucier, R.T. (1982). "Engineering geology of New Orleans." Geological Society of America, Reviews in Engineering Geology, 5, 75-93.
- Kolb, C.R., and Van Lopik, J.R. (1958). *Geological Investigations of the Mississippi River-Gulf Outlet Channel*, Miscellaneous Paper No. 3-259, U.S. Army Engineers Waterways Experiment Station, Vicksburg LA.
- Lemmon, A.E., Magill, J.T., and Wiese, J.R., Eds., Hebert, J.R., Cons. Ed. (2003). *Charting Louisiana: Five hundred years of maps*. The Historic New Orleans Collection, New Orleans.
- Mansur, C. I., and Kaufman, R. I. (1956). "Underseepage, Mississippi River Levees, St. Louis District." *Journal of the Soil Mechanics and Foundations Division*, 82(1), 385-406.
- McPhee, J. A. (1989). *The Control of Nature*. Farrar, Straus, Giroux, New York.
- Meade, R.H., and Parker, R.S. (1985). "Sediment in Rivers of the United States." *National Water Summary 1984 – Water Quality Issues*, U.S. Geological Survey, 49-60.

- Mississippi River Commission (MRC), and Lower Mississippi Valley Division. (1975). *The Flood of '73*, Corps of Engineers, U.S. Army.
- Moat, L.S. (1896). *Frank Leslie's Famous Leaders and Battle Scenes of the Civil War*, Mrs. F. Leslie, New York.
- Moore, N. R. (1972) *Improvement of the Lower Mississippi River and Tributaries 1931-1972*, Department of the Army, Corps of Engineers, Mississippi River Commission, Vicksburg MS.
- Morgan, A. E. (1971). *Dams and Other Disasters: a Century of the Army Corps of Engineers in Civil Works*, Porter Sargent Publishers.
- Noble, C. C. (1976). "The Mississippi River Flood of 1973." *Geomorphology and Engineering*, D.R. Coates (Ed.), Dowden, Hutchinson & Ross, Halsted Press.
- Press, F., and Siever, R. (1997). *Understanding Earth*, 2nd Ed., W.H. Freeman & Co.
- Saucier, R.T. (1963). *Recent Geomorphic History of the Pontchartrain Basin. Louisiana, Coastal Studies Series No 9*, Louisiana State University Press, Baton Rouge LA.
- Saucier, R.T. (1994). *Geomorphology and Quaternary Geologic History of the Lower Mississippi Valley*, U.S. Army Corps of Engineers Waterways Experiment Station, Vicksburg, MS.
- Shallat, T. (1994). *Structures in the Stream: Water, Science, and the Rise of the U.S. Army Corps of Engineers*, University of Texas Press.
- Shallat, T. (2000). "In the Wake of Hurricane Betsy." *Transforming New Orleans and Its Environs: Centuries of Change*, C.E. Colten (Ed.), University of Pittsburgh Press, Pittsburgh, 121-137.
- Snow, R.F. (1992). "Low and Dry." *American Heritage Invention & Technology*, 8(2), 4-5.
- Watson, C.C. (1982). An assessment of the lower Mississippi River below Natchez, Mississippi. Ph.D. dissertation, Department of Civil Engineering, Colorado State University, Fort Collins, Colorado. 162 p.
- Williams, F. E. (1928). "The Geography of the Mississippi Valley." *Great Inland Water-Way Projects in the United States*, Annals of American Academy of Political and Social Science, CXXXV (January), 7-14.
- U.S. Army Corps of Engineers. (1965). *Hurricane Betsy September 8-11, 1965*, U.S. Army Engineer Office, New Orleans LA
- U.S. Army Corps of Engineers. (1984). *Lake Pontchartrain, Louisiana, and Vicinity Hurricane Protection Project. Reevaluation Study*. New Orleans District, New Orleans LA.
- U.S. Army Corps of Engineers. (1987). *Design Memorandum No. 17, General Design, Jefferson Parish Lakefront Levee*, Vol. 1, New Orleans District, New Orleans LA.
- Works Projects Administration of Louisiana. (1937). *Some Data in Regard to Foundations in New Orleans and Vicinity*, New Orleans LA. 243 p.

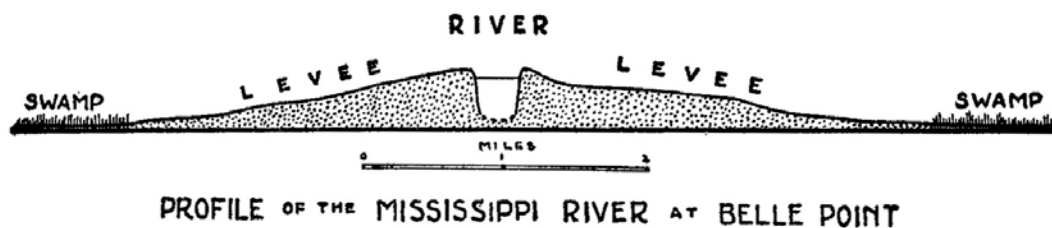


Figure 4.1: Typical cross section through the sandy bank levees of the Mississippi River, illustrating how the river's main channel lies above the surrounding flood plain, which were poorly drained swamp lands prior to reclamation in the post Civil War era (from Williams, 1928).

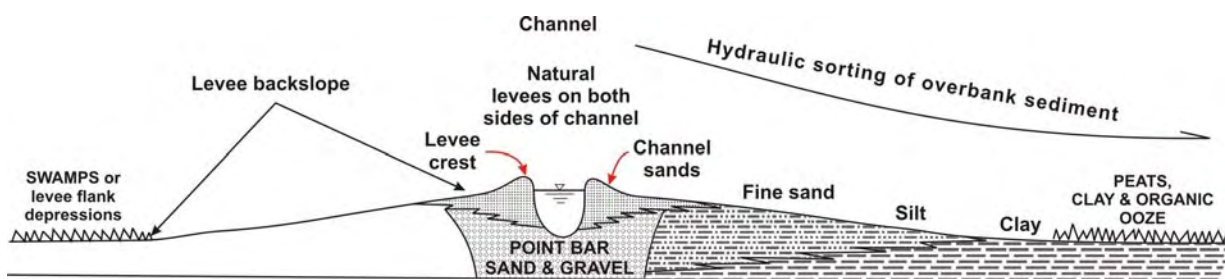


Figure 4.2: Same typical cross section, showing the hydraulic sorting of sediments moving away from the Mississippi River channel. The levee backslope zone lies between the elevated levees and the poorly drained swamps. In New Orleans, the Carrollton, Uptown, French Quarter, and Central Business Districts are situated on the natural levee and its backslope, while the Mid-City area was built on a levee flank depression between the Mississippi and Metairie levees. The Lakeview, Gentilly, and Ninth Ward areas occupy the old cypress swamps.

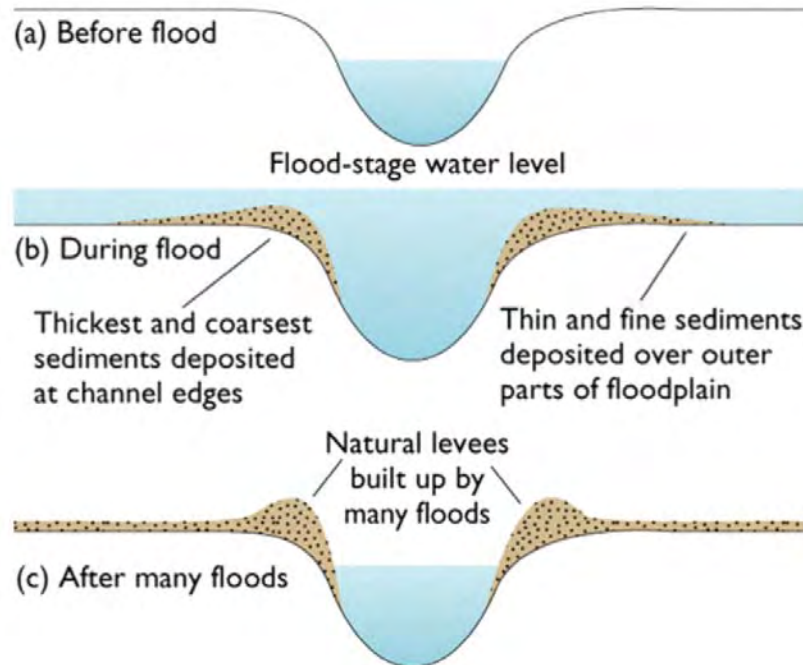


Figure 4.3: Natural levees exist along most perennial channels subject to periodic overbank flooding emanating from a prominent low flow channel, as sketched above. Man-made levees originated by piling up additional earthen fill on top of these natural levees (from Press and Siever, 1997).

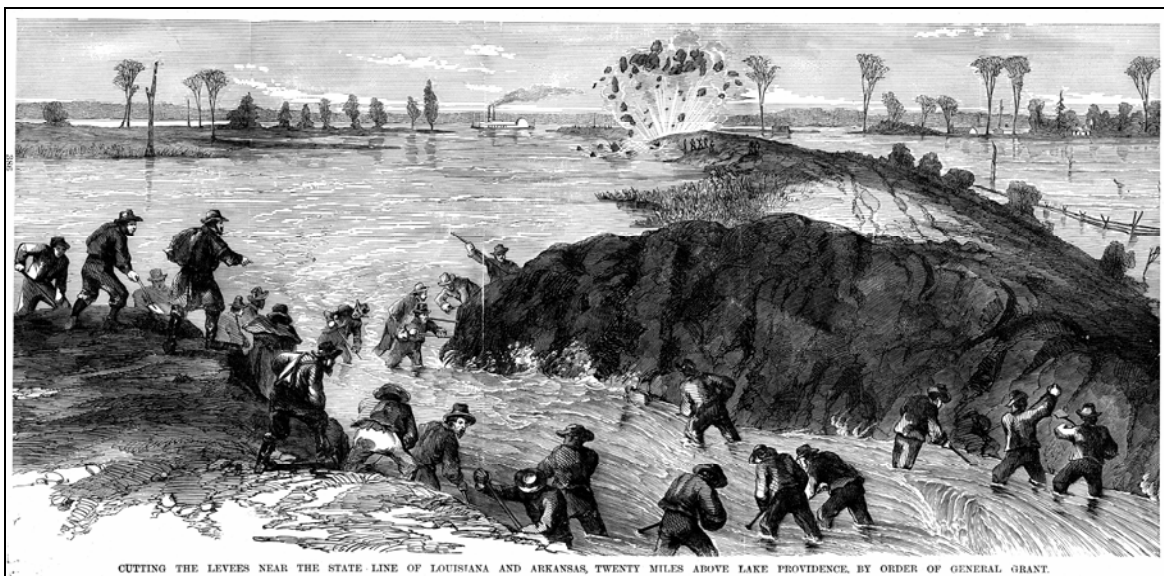


Figure 4.4: Union forces under General Grant cutting the levee near the state line of Louisiana and Arkansas, 20 miles above Lake Providence (from Moat and Leslie, 1896). In describing this activity, Moat and Leslie (1896) noted: “*The soil is very tough, and will not wash away. The levees consequently have to be blown up with gunpowder. The soil is then loosened with spades.*” Levees constructed of cohesive clay were found to be the most resilient, but those constructed of other materials, such as overbank silt, peat, or organic ooze were easily eroded.

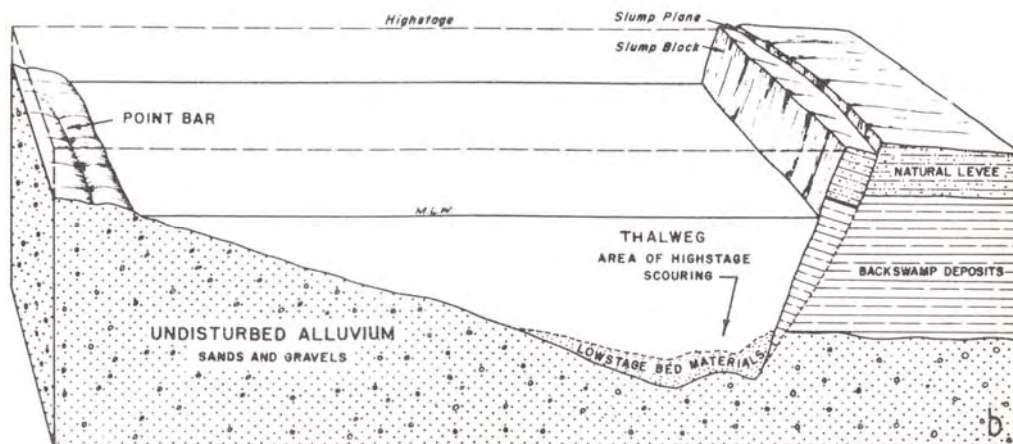


Figure 4.5: Asymmetric channel cross section typical of the Mississippi River, showing slumping of the oversteepened banks on the outside of its turns and the relative position of the river's thalweg, the line connecting the lowest points along the bed of the river. River mileage is measured along the thalweg, not along the river centerline, because this line more accurately describes the actual flow path (from Fisk, 1952).



Figure 4.6: Map showing the lands inundated in Louisiana during the height of the great Mississippi River Flood of 1927 (from the Historic New Orleans Collection). Concerns over long term safety from flooding caused many businesses and financial institutions to depart New Orleans to seemingly safer havens, such as Houston, TX (Barry, 1997).

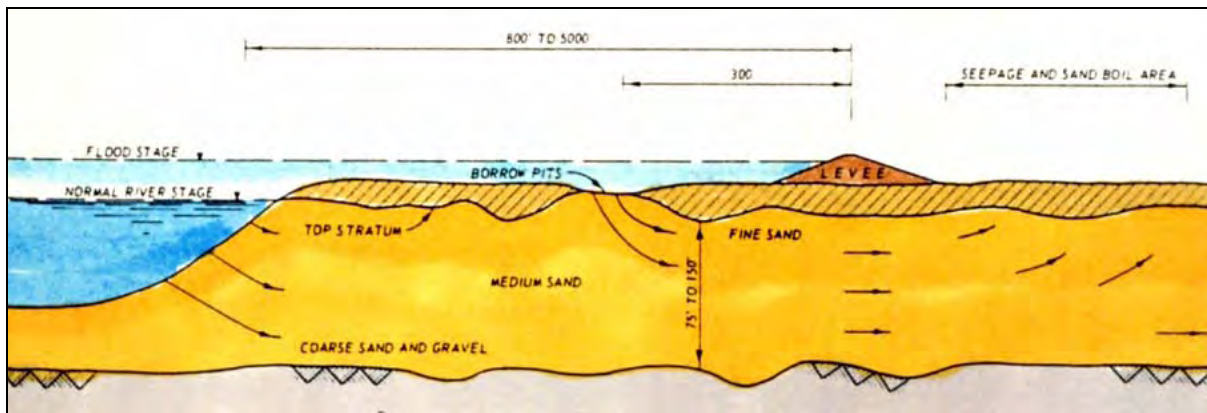


Figure 4.7 – Cross section through a typical Corps of Engineers levee in an alluvial valley (from Mansur and Kaufman, 1956). Analyses of levee stability depend in large measure on various assumptions made about seepage conditions beneath and adjacent to such structures. For instance, the coarse sand and gravel shown here may be 1000x more permeable than the overlying medium sand.

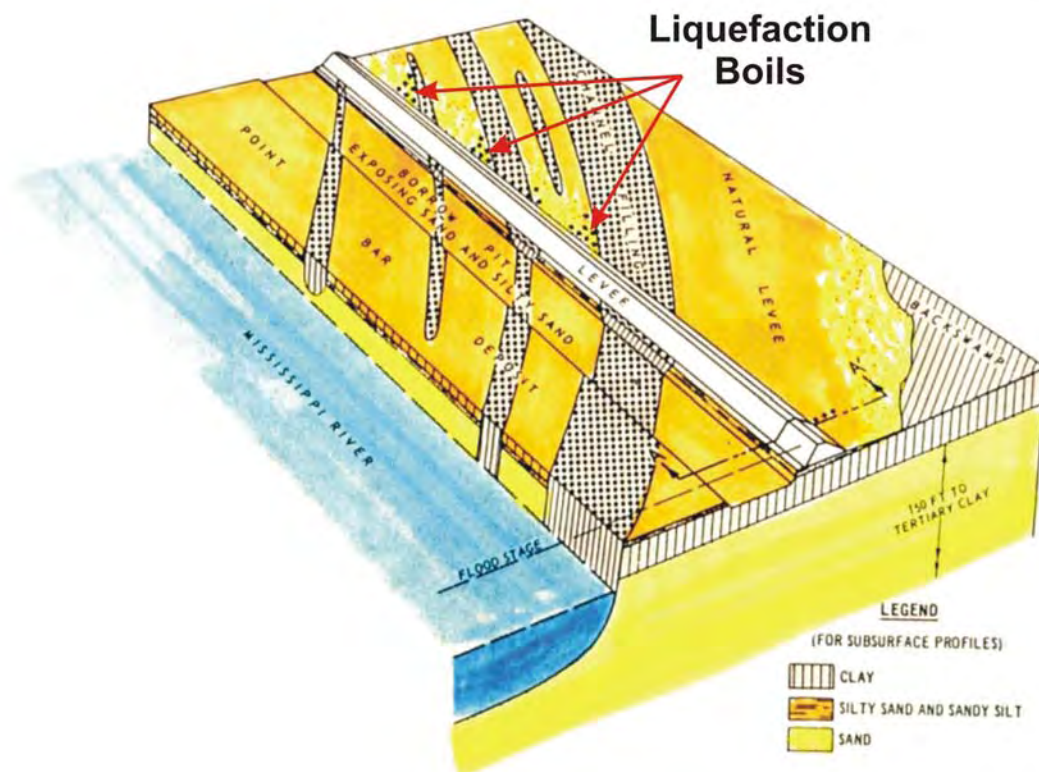


Figure 4.8: A major problem with man-made levees constructed during the MR&T Project is that they are necessarily constructed upon highly heterogeneous foundations, as portrayed here (taken from Kolb, 1976). The sharp contrast between highly organic channel fills (stippled zones) and natural levee sands and gravelly point bars promotes dangerous concentrations of seepage and differential settlement.

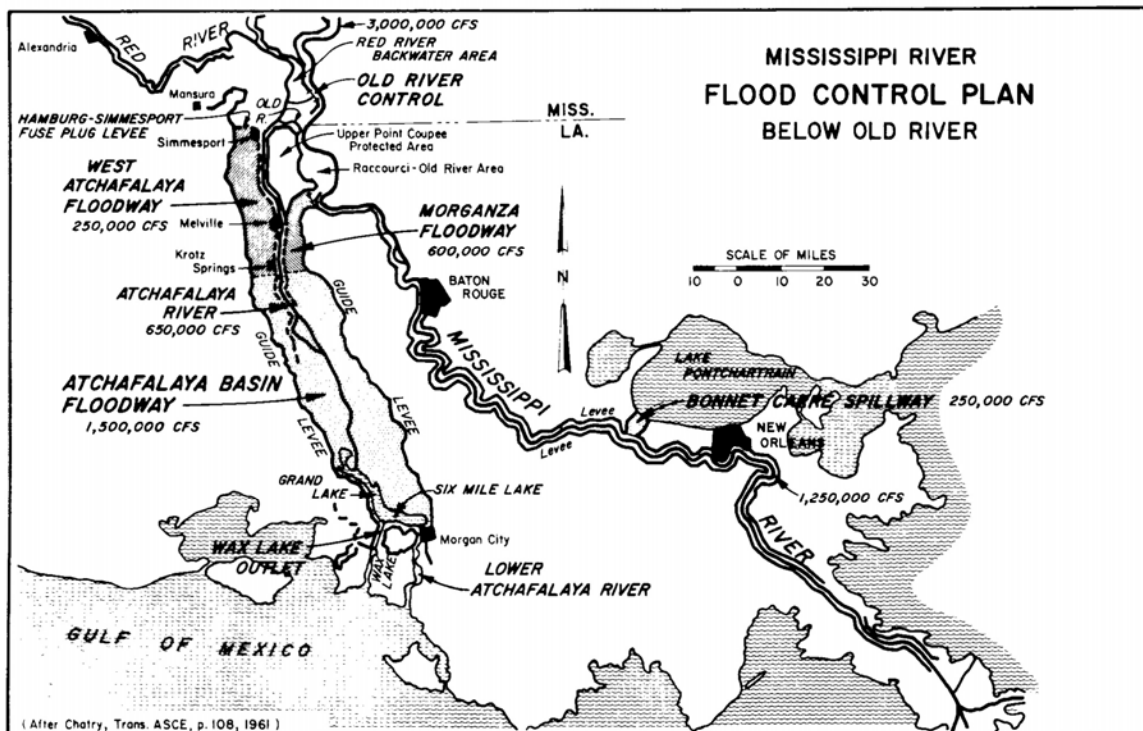


Figure 4.9: Map showing principal elements of the Mississippi River & Tributaries Project flood control for the lower Mississippi River Delta region (from Chatry, 1961). Note the much shorter flow channel to the Gulf of Mexico along the Atchafalaya River as opposed to the Mississippi River. The Mississippi River would have switched to this channel by 1975 if the Old River Control Structure had not been constructed in 1961-63.

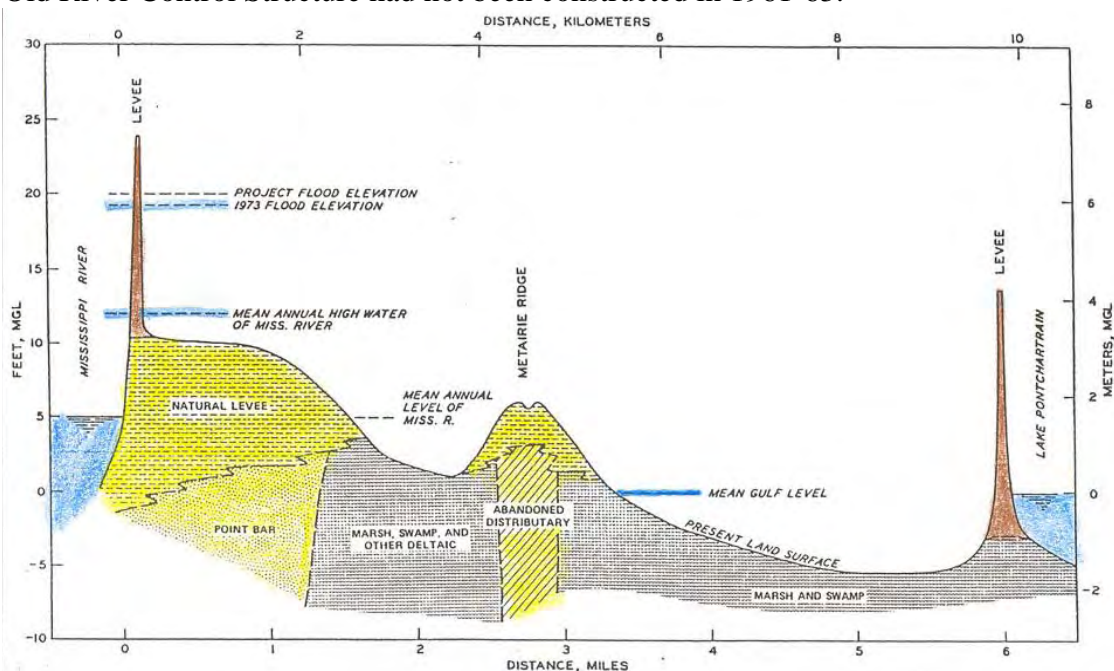


Figure 4.10 – This cross section illustrates how much of New Orleans lies below mean gulf level, requiring every drop of rain water to be pumped out. The height of the Mississippi River levee is +24.5 ft, MGL while the Lake Pontchartrain levee crests at +13.5 ft, MGL (from Kolb and Saucier, 1982).

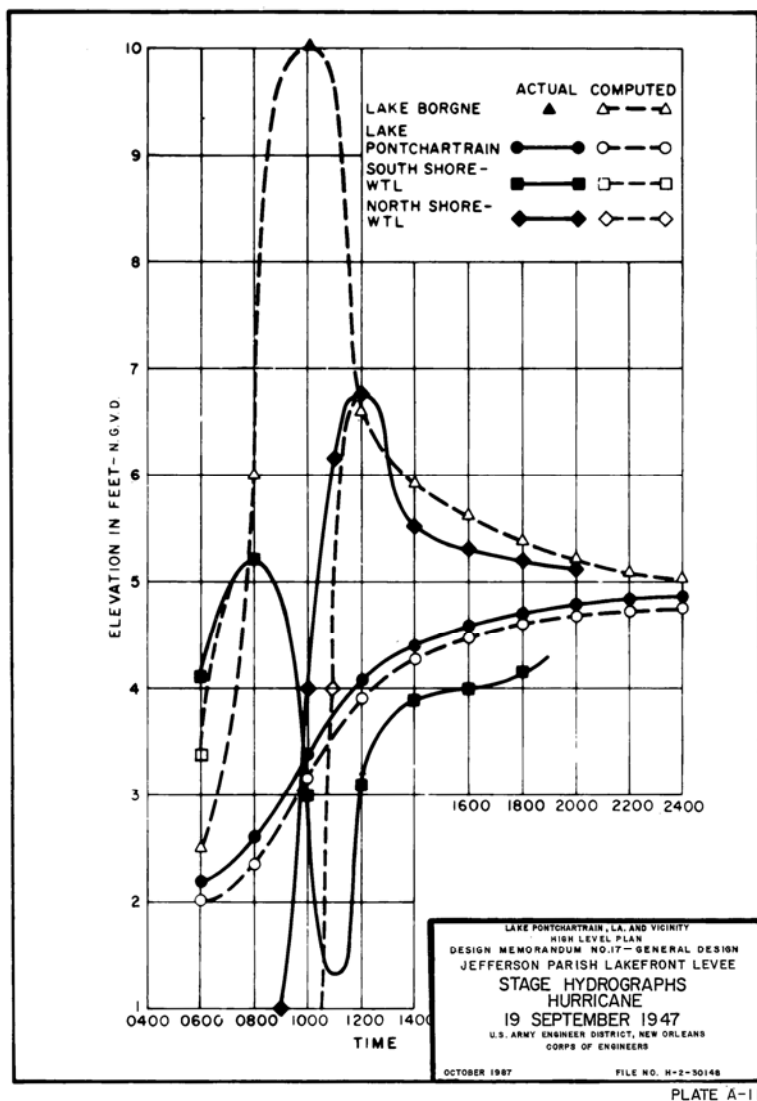


Figure 4.11: Stage hydrographs on Lakes Borgne and Pontchartrain from the September 1947 hurricane (from USACE DM-17, 1987). The 10 foot surge on Lake Borgne was the highest recorded value up to that time, though short-lived. A 13 foot surge was reported along lake Pontchartrain during the 1915 Grand Isle Hurricane, but this was before storm surge recorders were emplaced along the shorelines.



Figure 4.12: Portion of the flood inundation map from Hurricane Betsy in 1965, showing the areas on either side of the Inner Harbor Navigation Channel which were affected by overtopping, from storm surges on Lakes Borgne and Pontchartrain (from USACE, 1965).

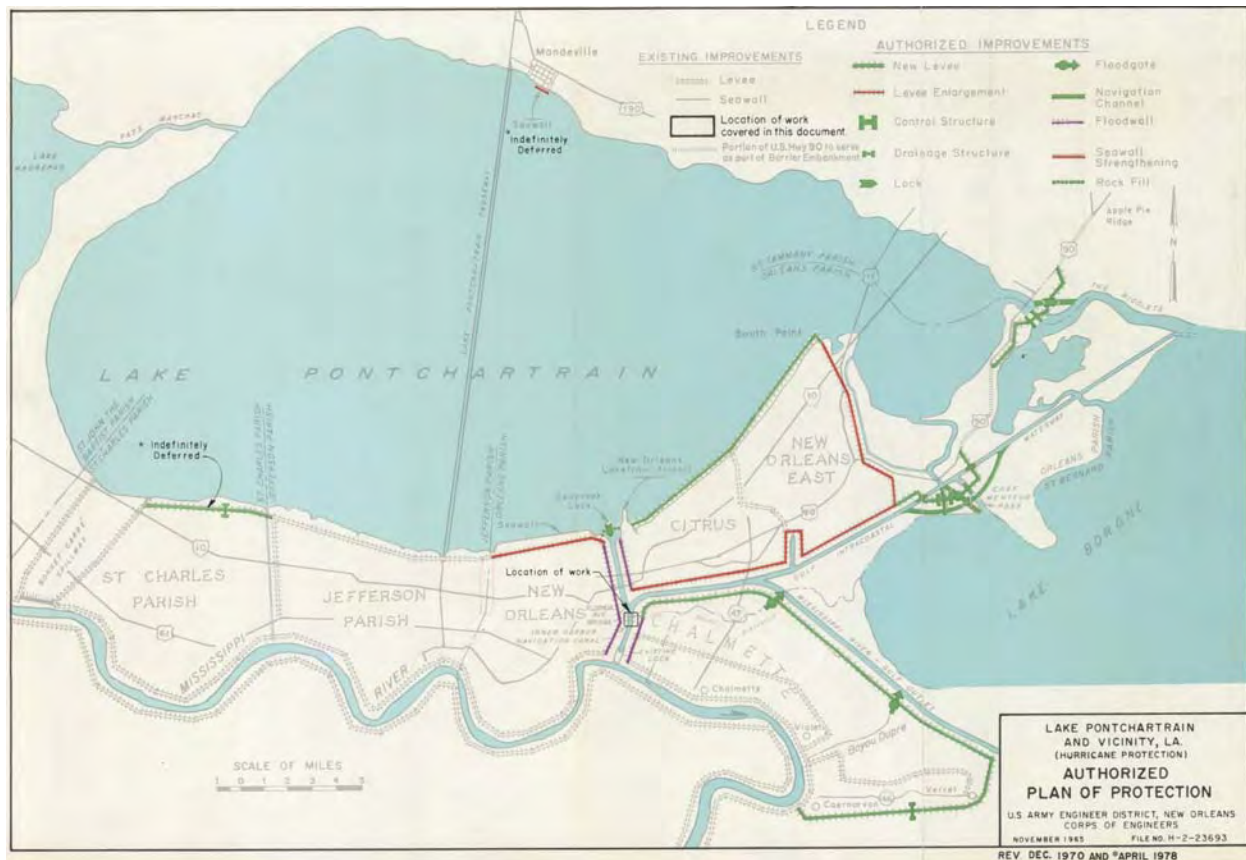


Figure 4.13: South Lake Pontchartrain flood protection measures authorized by Congress in the wake of Hurricane Betsy in 1965. These included heightening of the protective levees along the IHNC and the Lake Pontchartrain shoreline to the Orleans-Jefferson Parish boundary, and around Chalmette in St. Bernard's Parish. This system was subsequently enlarged to include the Pontchartrain levee all the way to the Bonne Carré Spillway and along the principal drainage canals in New Orleans and Jefferson Parishes.

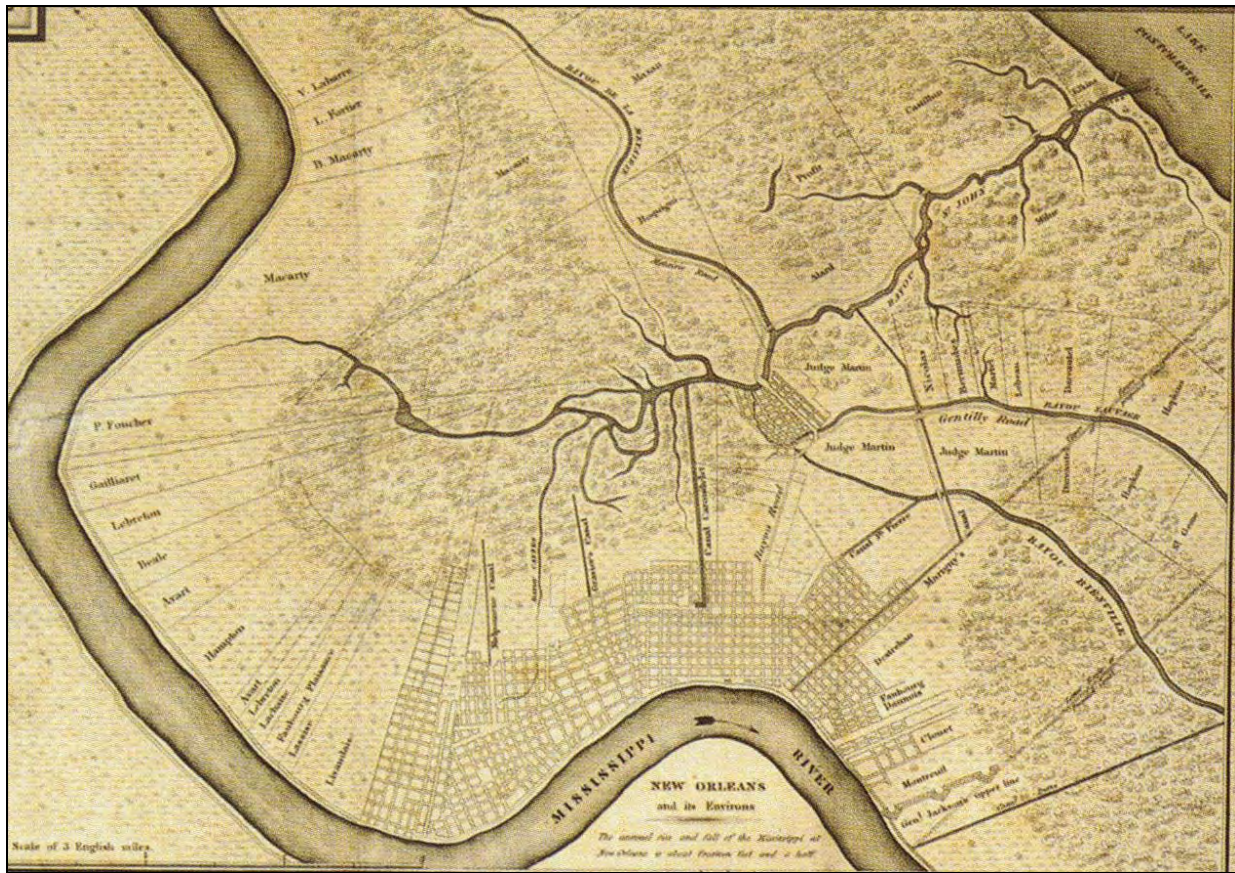


Figure 4.14: Plan of the City of New Orleans prepared by Francis Ogden in 1829. Note the linear drainage canals feeding into Bayou St. John, thence into Lake Pontchartrain (from the Historic New Orleans Collection).

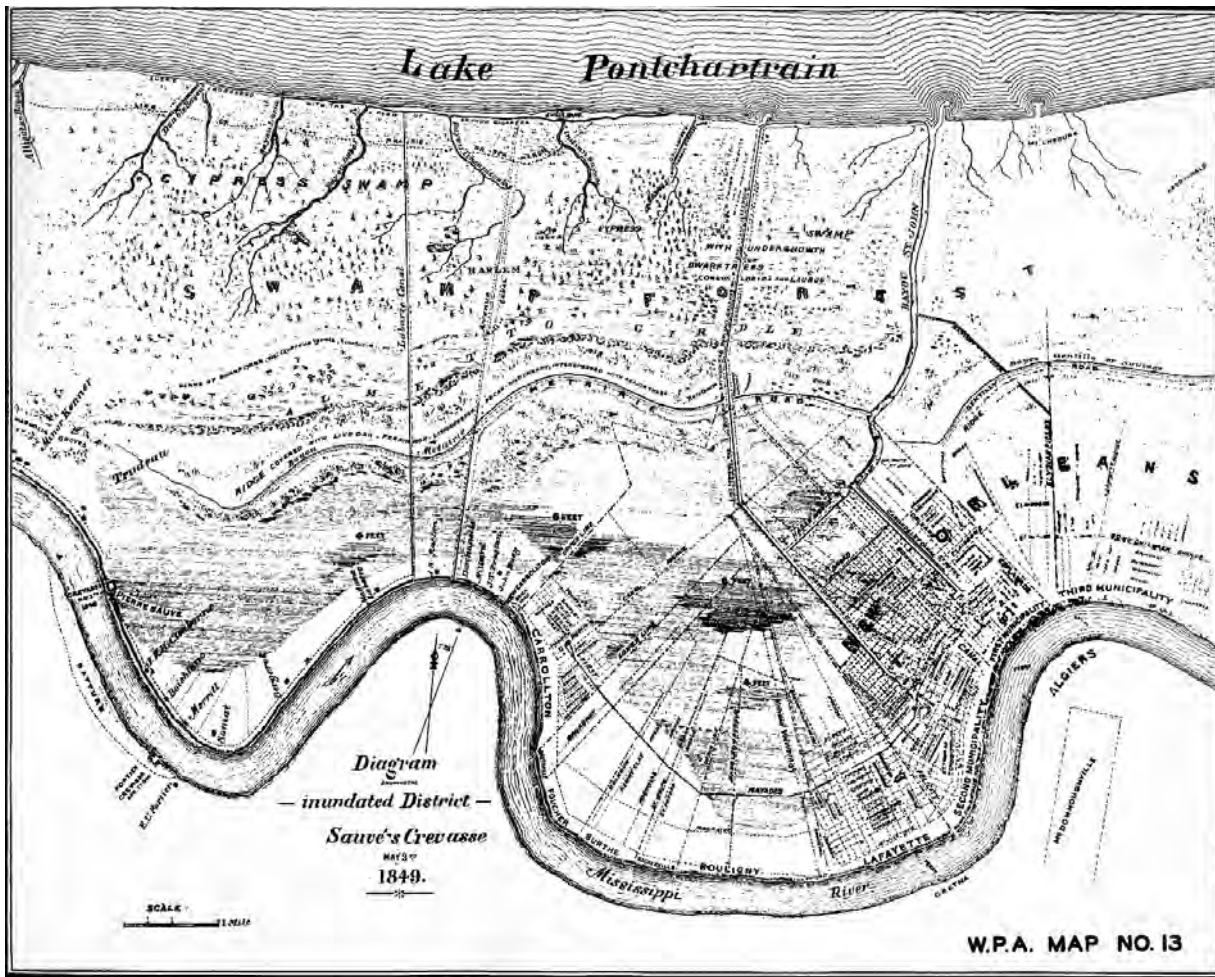
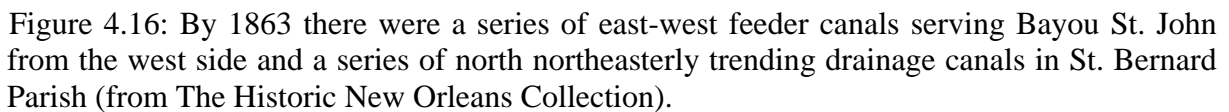


Figure 4.15 - Map of Sauv s Crevasse and the portions of New Orleans inundated by the flooding of 1849, the last significant flood to affect the city emanating from the Mississippi River. This 1849 map shows the extensive cypress swamps lying between the uptown and French Quarter areas and Lake Pontchartrain. The Carondelet and New Orleans Canals are clearly shown, but curiously omits the New Basin Canal (built in the 1830s). The map clearly shows the projected path of the 17th Street Canal between Orleans and Jefferson Parishes, suggesting it was being proposed (it appears to have been completed in 1857-58). The Labarre Canal in Jefferson Parish (near today's Bonnabel Canal) was likely never built (taken from WPA, 1937).



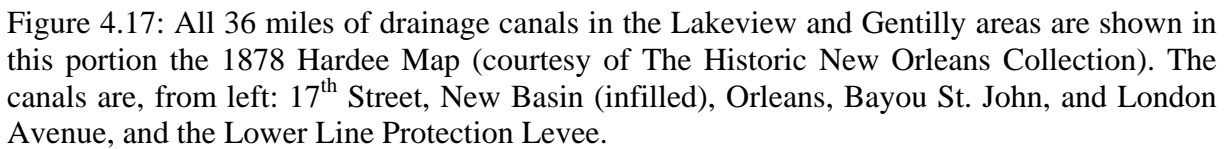




Figure 4.18 – Photo taken in 1890 looking north along the “shell road” than ran along the west side of the New Basin Canal, seen at extreme right. Note the modest height of the original embankment, no more than 5 feet above the adjacent cypress swamp at left. The original embankments were heightened after hurricane-induced overtopping in 1915 and 1947 (image from the University of New Orleans Special Collections, New Orleans Views).

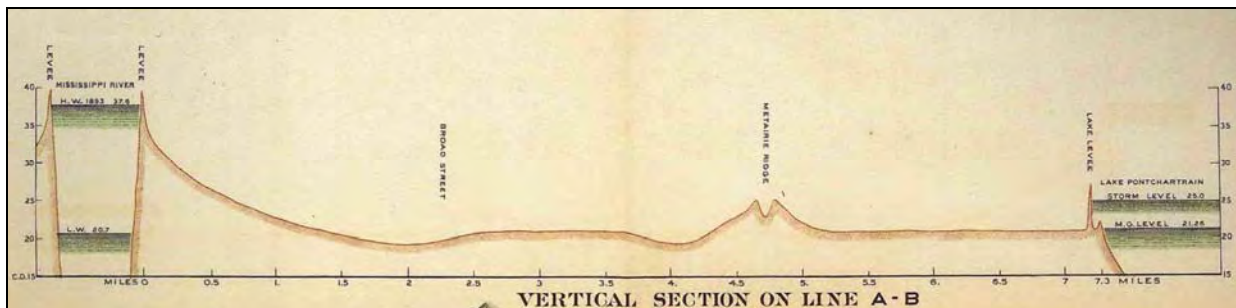


Figure 4.19: Cross section through New Orleans prepared by City Engineer L. W. Brown in 1895 (from the Historic New Orleans Collection). This shows the elevated position of the Mississippi River and the Metairie-Gentilly Ridge distributary channel, which lies 3 to 6 feet above the surrounding area. The green lines denote high and low levels in the river and Lake Pontchartrain. Elevations are in the old Cairo Datum (21.26 ft above MGL).

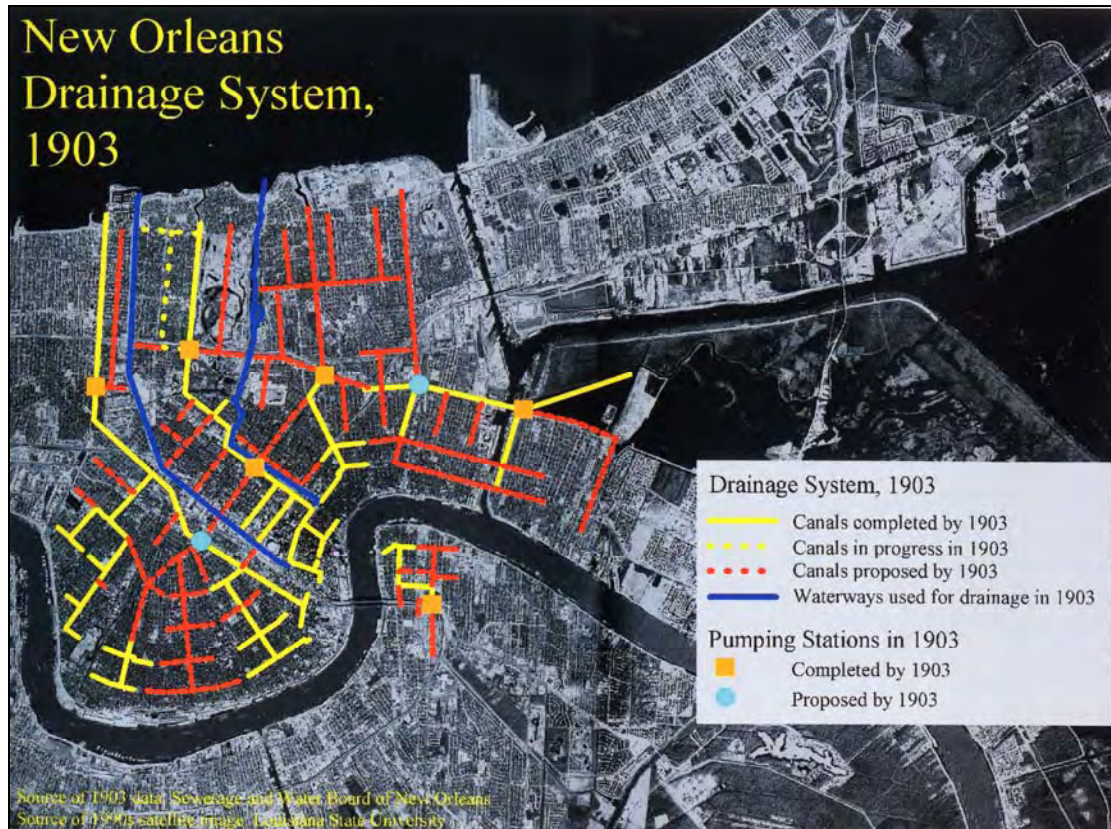


Figure 4.20: Principal elements of drainage system infrastructure as it existed in 1903 (taken from Campanella, 2002). The 17th Street and London Avenue Canals had already been in operation for several decades.



Figure 4.21: S&WB engineer A. Baldwin Wood standing next to one of his 14-foot diameter screw pumps in 1929 with several of the board's secretaries sitting inside the housing for scale (courtesy of the Sewerage & Water Board of New Orleans).

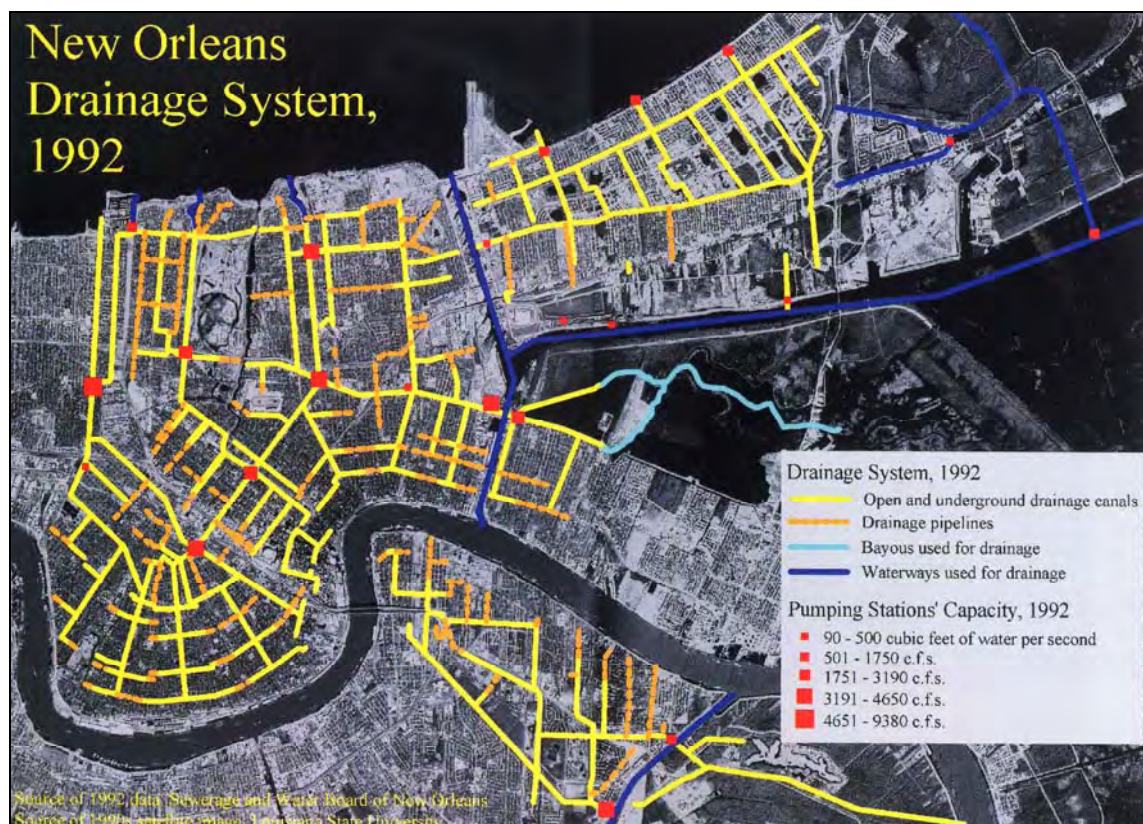


Figure 4.22: Principal elements of the pre-Katrina drainage system infrastructure as it existed in 1992 (taken from Campanella, 2002). The aggregate pump capacity could have cleared the city of flood waters in less than three days if the levees had simply been overtopped without failing.

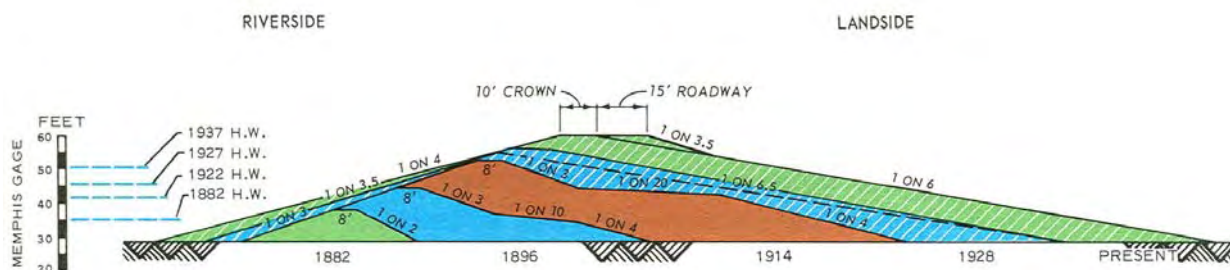


Figure 4.23: Evolution of the Corps of Engineers' standard levee section, 1882 to 1972 (from Moore, 1972). Earth embankment levees are generally heightened sequentially by compacting additional soil on the land side of the embankments (each sequence of heightening shown as different colors). Levees adjacent to drainage canals or perennial channels are not raised on the river side of the embankment because excess moisture would prevent meaningful compaction of the fill. Existing homes abutted the landside of the drainage canal levees in New Orleans by the time the Corps of Engineers began analyzing them in the 1960s.



Figure 4.24 (upper): View looking up the east side of the London Avenue Canal near Robert E. Le Boulevard crossing showing the encroachment of homes against the slope of the levee. This situation was common across New Orleans (photo by C. M. Watkins).



Figure 4.24 (lower): Concrete flood wall along the west side of the 17th Street Canal in Jefferson Parish, where a street runs along the toe of the embankment. This scene is typical of the concrete I-walls constructed on steel sheetpiles driven into the crest of the drainage canal embankments in New Orleans in the 1990s to provide additional flood freeboard from hurricane-induced storm surges (photo by J. D. Rogers).

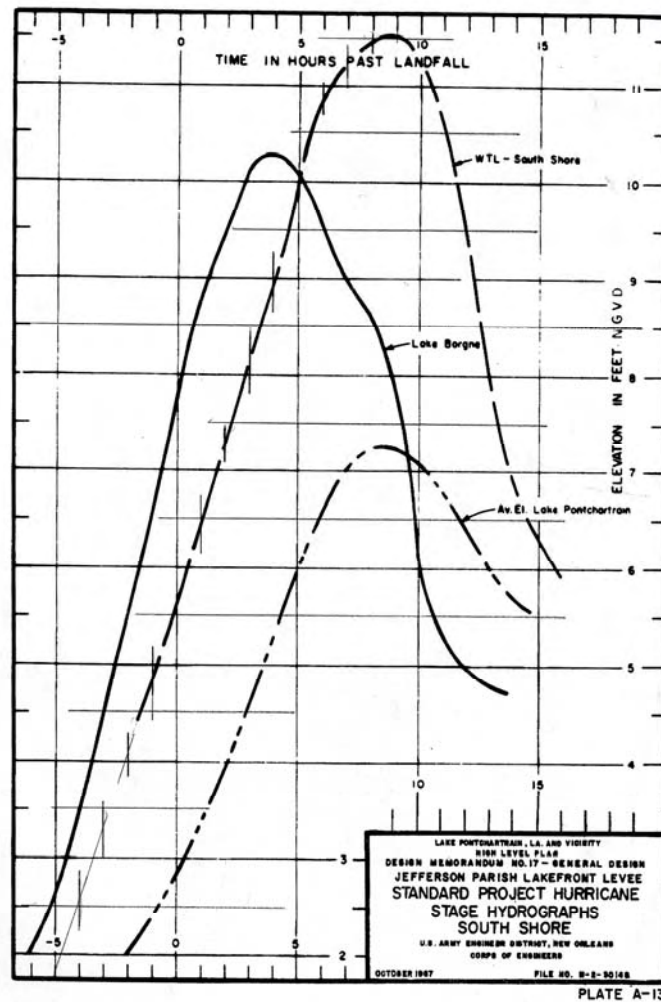
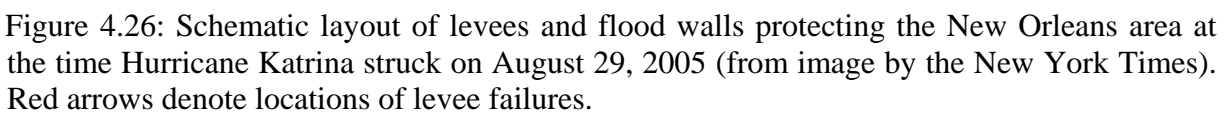


Figure 4.25: Assumed Category 3 storm surge curves for the Gulf of Mexico shoreline, Lake Borgne, and Lake Pontchartrain used by the Army Corps of Engineers for planning and design purposes prior to Hurricane Katrina in 2005. Note the short duration of extreme surges, about 12 hours duration above 5 ft MGL for Lake Pontchartrain (taken from USACE DM-17, 1987).



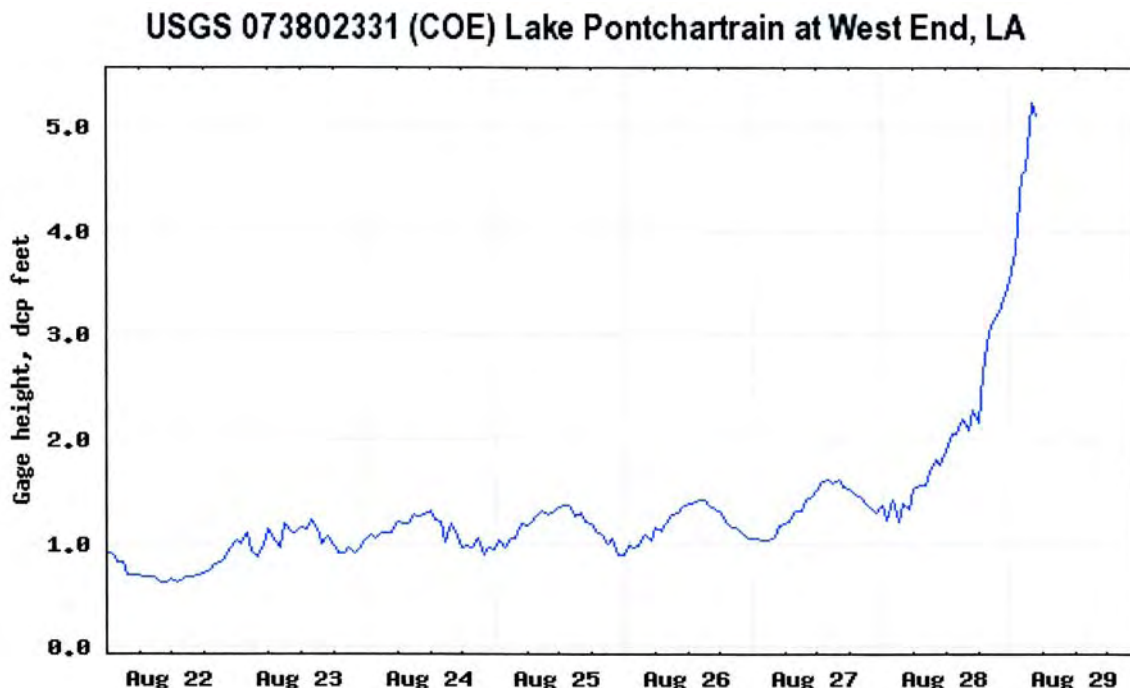


Figure 4.27: Incomplete record of the Lake Pontchartrain tidal stage gage at West End, near the mouth of the 17th Street Canal during the early stages of Hurricane Katrina (from U.S. Geological Survey). This record shows the progressive building of tidal stages, days before the storm made landfall. Significant “ramping” of the storm surge began on August 28th, with the sharpest increase on the morning of August 29th, when the hurricane made landfall. The gage failed when the lake level reached 5.3 ft, before the peak surge was recorded.



Figure 4.28: Mobile dragline constructing the Morrison-Picayuneville Levee about 25 miles south of New Orleans in June 1931 (from Elliot, 1932). Tower draglines could excavate materials up to a quarter mile away, dragging it back up onto the new levee.



Figure 4.29: Aerial oblique view of the Inner Harbor Navigation Canal between 1960-64, after the entry to the Mississippi River-Gulf Outlet Channel had been enlarged (upper right), connecting to the inner harbor area (photo from the Army Corps of Engineers).



Figure 4.30: Seepage crevasse splay exposed on the water side of the east levee of the IHNC breach, likely from back-drainage of the flood-inundated area of the Lower Ninth Ward, after Hurricanes Katrina and Rita. This same section of the IHNC levee failed in 1965 during Hurricane Betsy. Seepage crevasse splays tend to occur where high permeability materials daylight, or come into proximity of the existing ground surface. They are easily recognized by anomalous seepage and the birdfoot pattern of the splays, which are often filled with soils displaced by seepage and hydraulic piping. (photo from U.S. Army Corps of Engineers).

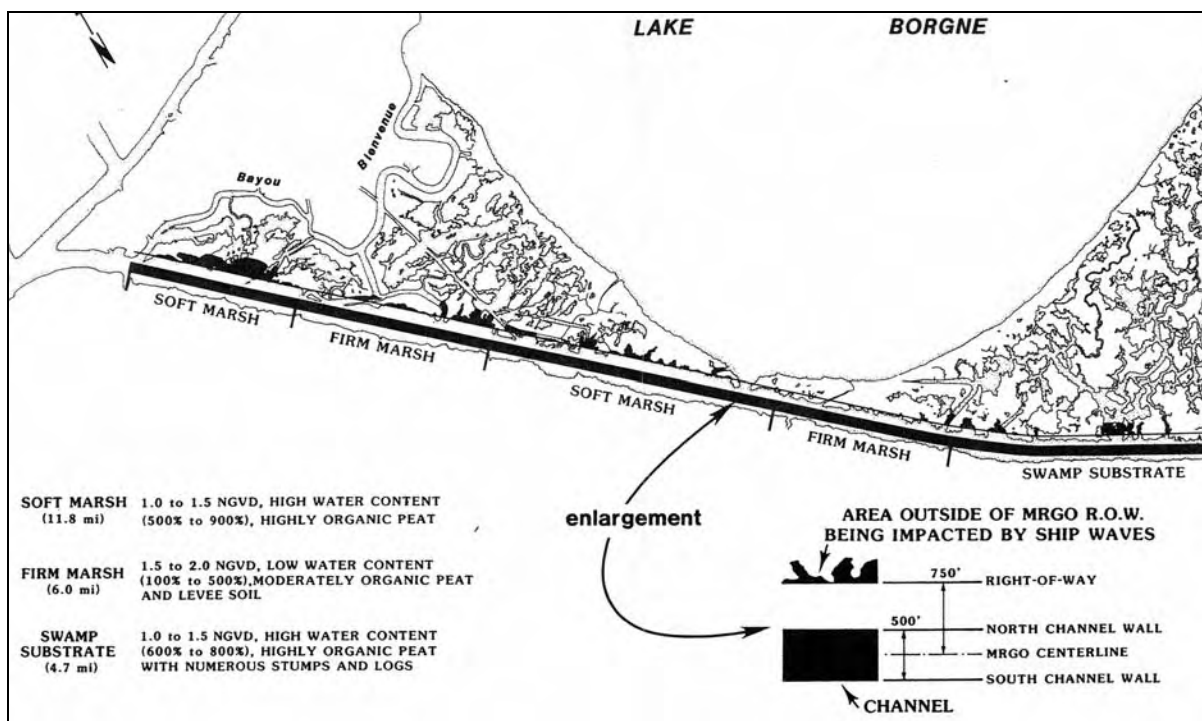
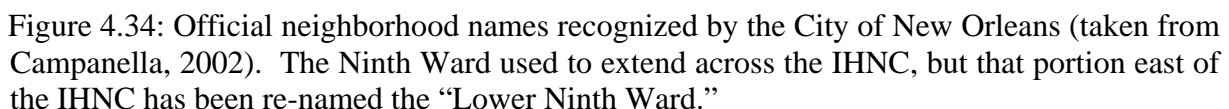


Figure 4.31: Portion of a map of the upper MRGO channel adjacent to Lake Borgne from the report by Coastal Environments, Inc. (1984). This shows the major soil subdivisions they identified: soft marsh, firm marsh, and swamp substrate. Much of this material was unsuitable for using in the adjoining levee embankments.



Figure 4.32: Area where the southwest bank of the MRGO channel levee within two miles southeast of Bayou Dupree was completely swept away by overtopping from Lake Borgne (photo by L. F. Harder).

Figure 4.33: Table relating correction factors used when comparing various historic datums in the New Orleans area (Denny, 2002). Blanket corrections can no longer be made to adjust elevations to NAVD88-2004.65, which is the most oft cited datum currently used in New Orleans. The reason for these disparities is the gross differential settlement between reference benchmarks, which can be significant (order of magnitude difference).



CHAPTER FIVE: THE LOWER MISSISSIPPI REGION AND PLAQUEMINES PARISH

5.1 Overview

Plaquemines Parish is the area where the last portion of the Mississippi River flows out into the Gulf of Mexico (see Figures 2.6 and 5.1). Extending southeast from New Orleans, Plaquemines Parish straddles both sides of the lower reaches of the Mississippi River for about 70 miles out to the river's mouth in the Gulf. This protected strip, with "river" levees fronting the Mississippi River and a second, parallel set of "storm" levees facing away from the river forming a protected corridor less than a mile wide, serves to protect a number of small communities as well as utilities and pipelines. This protected corridor also provides protected access for workers and supplies servicing the large offshore oil fields out in the Gulf of Mexico.

It is an area that is sparsely populated, with a population of only about 27,000 people in the entire parish just prior to Hurricane Katrina's arrival (see Plaquemines Parish Government Website: <http://www.plaqueminesparish.com>). Most of these people live in small, unincorporated towns and villages along the river. Not only are these communities subject to potential flooding from the Mississippi River, but they are also vulnerable to flooding from hurricane surges because the parish extends so far out into the Gulf from the mainland.

For flood protection from the Mississippi River, large federal project levees were constructed along both sides of the river with design crest elevations of approximately +25 feet (MSL). For many of the communities lying closely alongside the Mississippi River levees, "hurricane" or back levees were also constructed behind them to protect them from hurricane surges coming from the Gulf. These hurricane levees were constructed with lesser crest heights than the river levees, and typically had crest heights on the order of +17 to +18 feet (MSL). Thus, many of the homes in these areas are sandwiched between two sets of levees: one along the river and the other behind the towns.

The Independent Levee Investigation Team was not able to devote significant time to detailed investigations and analyses of the numerous individual levee failures that occurred along this protected corridor. Accordingly, this chapter will present only a brief overview of the performance of the flood defenses in this parish during Hurricanes Katrina and Rita.

As described previously in Chapter 2, Plaquemines Parish was the first developed area to be severely affected by the large onshore storm surge as Hurricane Katrina approached the southern coast in the early morning of August 29, 2005.

Hurricane Katrina devastated many of the Plaquemines Parish communities. Hurricane Katrina was reported to have induced storm surges on the order of up to 20 feet in this region, as shown in Figure 5.2. In addition, large storm waves atop this surge rose to greater heights. This storm surge, and the waves that accompanied it, overtopped and damaged many portions of the "storm" levees. Both the United States Army Corps of

Engineers (see Figures 2.6 and 5.1) and the Plaquemines Parish Government website report numerous breaches of the storm levees and widespread deep flooding and destruction.

Figures 5.3 through 5.12 show examples of the types of damage and flooding that resulted from the overtopping and breaching of the protective hurricane levees.

Figure 5.3 shows an aerial view of the inundation of the hamlet of Myrtle Grove, on the west side of the Mississippi River, as it appeared on September 25, 2005, one day after the second Hurricane (Rita) again inundated this section.

Figure 5.4 shows an aerial photograph of a levee breach of the hurricane (back) levee on the western side of the Mississippi River near the community of Sunrise. The breach occurred at a “transition” between an earthen levee section with a sheetpile-supported concrete I-wall, and a plain structural floodwall section. Failures at transitions between different adjoining sections were relatively common throughout the affected area during Hurricane Katrina.

Figure 5.5 shows an aerial photograph of a breach of the hurricane (back) levee at another “transition” near the Hayes Pump Station. This time the failure occurred at a sheetpile transition between an earthen embankment and a structural floodwall section, and sheetpile to earthen embankment connection appears to have been the weak link.

Figure 5.6 shows a pair of large shrimp boats on Highway 23, near the foot of the Empire High Rise Bridge. As illustrated by this photo, overtopping was quite severe, and large objects were floated up onto, and sometimes over, the levees.

5.2 Point a la Hache

Point a la Hache is the parish seat for Plaquemines Parish and is located along the east side of the Mississippi River. Storm surges from the east largely overwhelmed the back levee, breached it in several places, and inflicted deep flooding and widespread destruction in this town. Figure 5.7 presents an aerial photograph of one such breach taken on September 25, 2005 (from Plaquemines Parish Government Website). Shown in this photograph is a temporary road constructed across the interim breach repair to facilitate access and repairs.

Figure 5.8 shows this same levee breach a few weeks later during the installation of a sheetpile cutoff that was undoubtedly intended to be part of an interim, and perhaps permanent repair. The team members viewing the installation believed that the sheetpile wall was a good concept to affect a positive cutoff of seepage through the deeply scoured breach and loose debris. However, during the installation, team members noted that the contractor was having difficulty advancing the southern portion of the sheetpiles very far into the ground using the equipment in use at the time of the team’s visit. It is hoped that the pilings ended up being driven to their needed depths.

Residences in Pointe a la Hache were commonly inundated to depths of 12 to 18 feet (see Figure 5.9). Inundation flooding was so great that water flowed across the community from the east towards the Mississippi River, and even overtopped the Mississippi River levee

(at least with significant wave splashover) by several feet. Based on debris found on tractor equipment left on the levee crown along the Mississippi River, overflows or splashover of up to 4 feet were estimated. For most of the areas visited by our team, relatively little significant damage was observed on the Mississippi River levees, possibly because the river sides of the levees viewed by the team were paved with concrete slope protection (see previous Figure 2.17). Damage to the “storm” levees was significant at many locations, however,

Like many New Orleans residences, the small wooden homes in Pointe a la Hache were commonly founded on cinderblock piers. As a result of the deep flooding and the flow towards the Mississippi River, homes in Pointe a la Hache were commonly picked up and floated away from their foundations. Many ended up being deposited on or across the Mississippi River Levee as a result of storm surges flowing from the overtopped “storm” levees towards the “river” levees alongside the Mississippi River (see Figures 5.10 through 5.12).

5.3 Erosion Studies

Although overtopping caused numerous breaches in the “storm” levees facing away from the Mississippi River, less erosion was observed along most of the Federal “river” levees. This may have been due in part to the fact that the river-side levee embankments slope faces were paved with concrete slope face protection (as shown previously in Figure 2.17, which clearly shows this river-side slope face protection.) It may also have been due in part to the fact that the backsides of these river levees, which had no formal slope face protection, were at least partially protected from the full energy of the storm surge and the wind driven waves by the obstacles presented by the “hurricane” levees, and by other obstructions including buildings and trees, etc.

Nonetheless, it is a noteworthy performance on the part of these levee embankments, and it merits further study. It is hoped that with further testing trends will emerge showing that soil type and character, as well as placement and compaction conditions, can be used as a relatively reliable basis for prediction of the level of vulnerability of levee embankment soils to erosion and scour. Issues associated with erosion are discussed in more detail in Chapter 9.

5.4 Summary

Plaquemines Parish is the most obviously exposed populated and flood protected area in the region. It juts out into the Gulf of Mexico much like a boxer’s chin, almost daring a knockout blow.

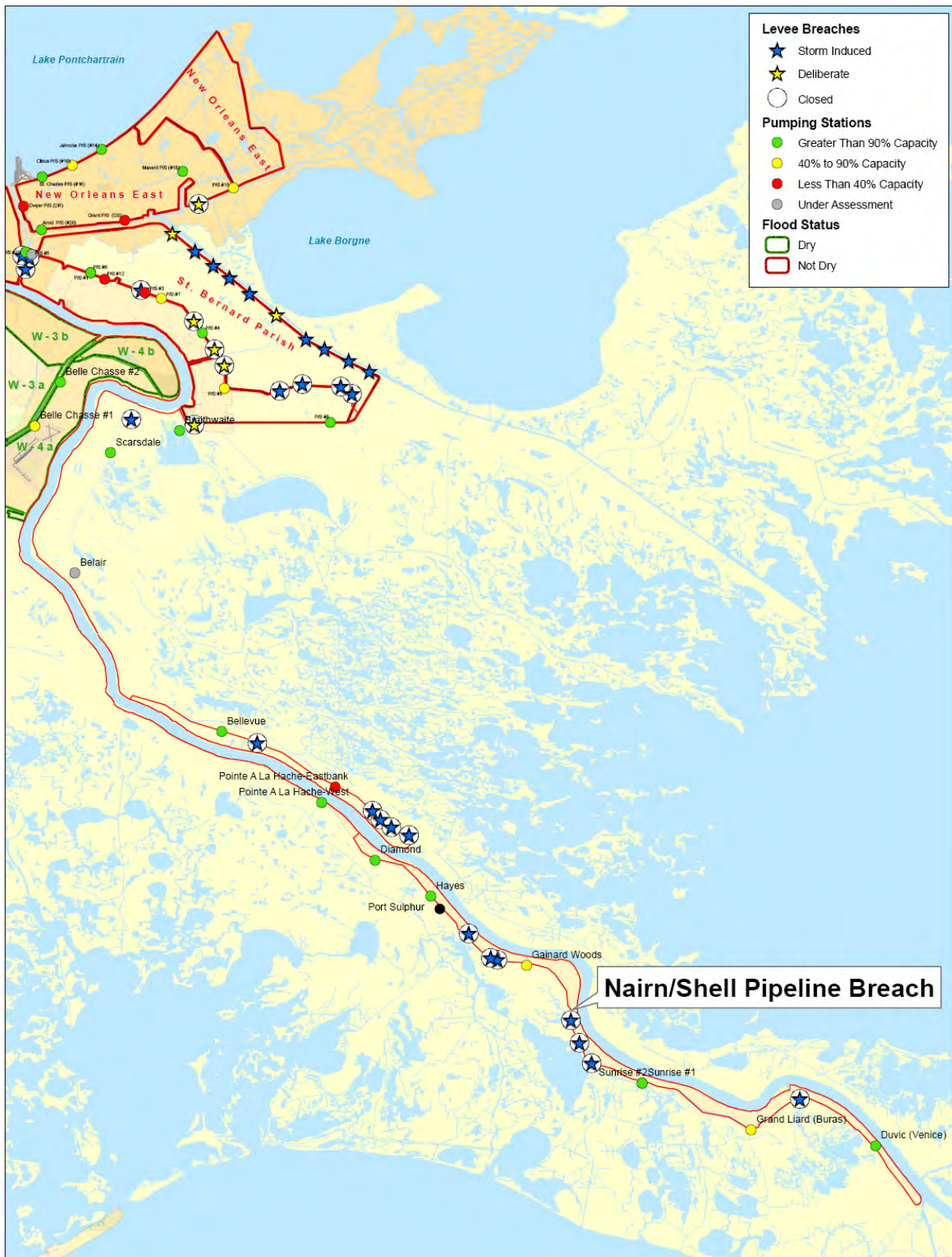
Because Plaquemines Parish is so obviously exposed, the evacuation of the Parish was unusually comprehensive prior to Katrina’s arrival. That was a good thing, as most of the lower reaches of the Parish were catastrophically flooded. Massive damage was done to homes and businesses in the many small and generally unincorporated townships, and there was at least one major rupture in an oil transmission line. The best information available to this investigation team at this time is that approximately 60 lives were lost in Plaquemines Parish during hurricane Katrina.

The merits of expending Federal dollars to attempt to defend the full Parish, or even large portions of it, in the face of ongoing regional subsidence, sea level rise, and increasing projected hurricane intensity due to rising Gulf water temperatures, warrant further study. Recent requests for up to \$3 billion in Federal funds to repair and upgrade the levees for a narrow strip of land into which less than 15,000 to 20,000 people are currently expected to return would represent an expenditure of approximately \$150,000 to \$200,000 per capita. In the mean time, large amounts of Federal funds are currently being expended to repair the damaged levees in this Parish.

5.5 References

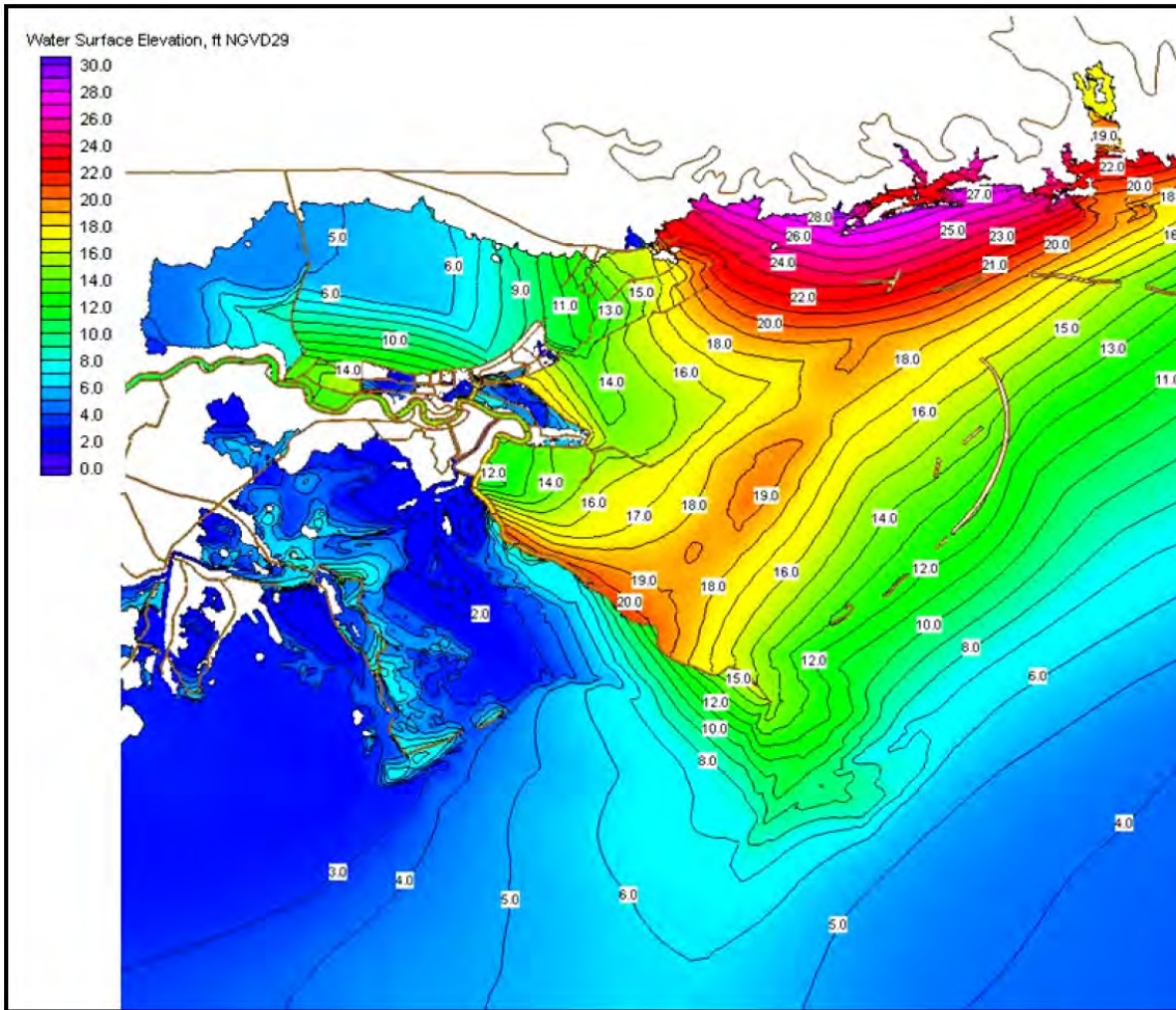
Interagency Performance Evaluation Task Force, (2006), "Performance Evaluation, Status and Interim Results, Report 2 of a Series, Performance Evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System," March 10, 2006.

Plaquemines Parish Website, (2006), <http://www.plaqueminesparish.com>



Source: Modified after USACE

Figure 5.1: Map showing the levee protected areas along the lower reaches of the Mississippi River (in the Plaquemines Parish Area).



Source: IPET (2006)

Figure 5.2: Aggregated maximum storm surge elevations (maximum among all times).



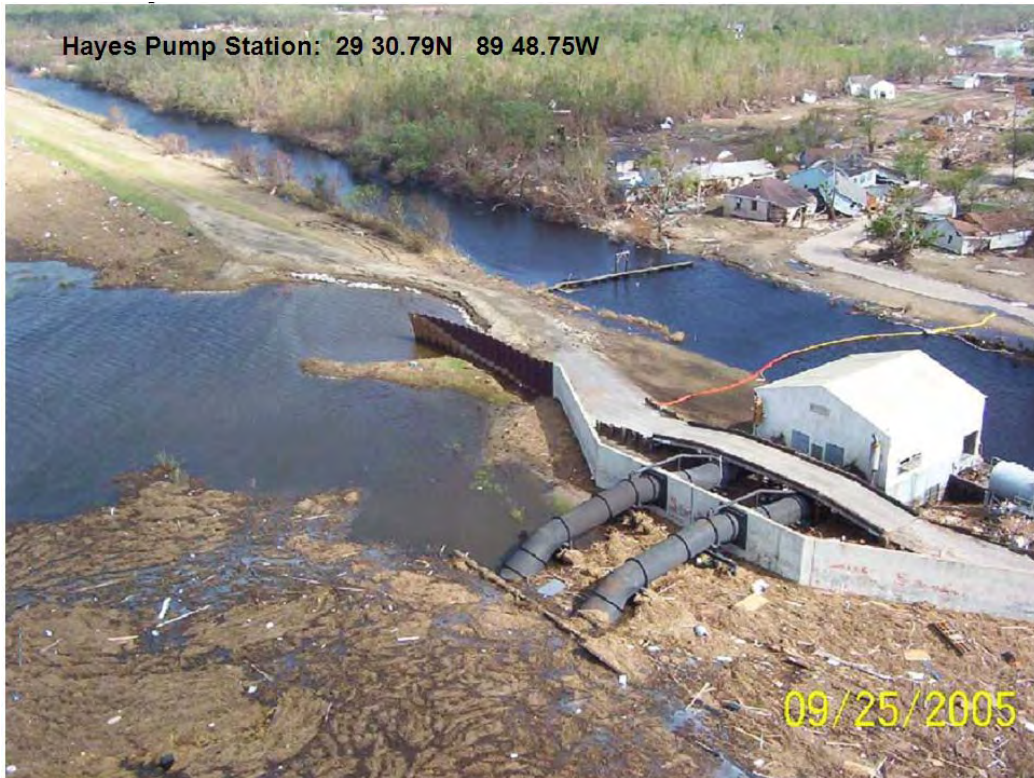
Source: <http://www.plaqueminesparish.com/>

Figure 5.3: Aerial photograph of inundated portion of Myrtle Grove along western side of the Mississippi River. [September 25, 2005]



Source: <http://www.plaqueminesparish.com/>

Figure 5.4: Aerial photograph of levee breach of storm (back) levee along western side of the Mississippi River near the community of Sunrise. [September 25, 2005]



Source: <http://www.plaqueminesparish.com/>

Figure 5.5: Aerial photograph of levee breach of storm (back) levee at levee-to-wall transition near Hayes Pump Station. [September 25, 2005]



Source: <http://www.plaqueminesparish.com/>

Figure 5.6: Aerial view of two large shrimp boats deposited on Highway 23 at the foot of the Empire High Rise Bridge.



Source: <http://www.plaqueminesparish.com/>

Figure 5.7: Aerial photograph of levee breach of storm (back) levee East of Pointe a la Hache. [September 25, 2005]



Photograph by Les Harder

Figure 5.8: Photograph of Sheetpile Cutoff Being Placed into Levee Breach of Storm (Back) Levee East of Pointe a la Hache. [October 12, 2005]



Photograph by Les Harder

Figure 5.9: Photograph of flood elevation on trees landward of hurricane levee East of Pointe a la Hache – illustrating that flood waters remained to large depths for extended periods. [October 12, 2005]



Photograph by Les Harder

Figure 5.10: Photograph of Pointe a la Hache home deposited on Mississippi River levee crown after storm surges overtopped the storm levee from the East (left) towards the River – which is to the right in this photograph.



Photograph by Les Harder

Figure 5.11: Photograph of Pointe a la Hache homes deposited on Mississippi River Levee after storm surges overtopped the levee from the East (left) towards the River (right). [October 12, 2005]



Photograph by Les Harder

Figure 5.12: Photograph of Pointe a la Hache home site where a wood home was floated off of its cinderblock piers. [October 12, 2005]

CHAPTER SIX: THE ST. BERNARD AND LOWER NINTH WARD PROTECTED AREA

6.1 Introduction

As described previously in Chapter 2, St. Bernard Parish and the Lower Ninth Ward are protected by a single continuous “ring” of levees that, together, constitute one of the three main protected basins flooded by hurricane Katrina.

Figures 2.11 and 6.1 show the locations of the principal breaches and distressed sections of the levee and floodwall system protecting this basin. Figure 6.2 shows the inundation of this basin four days after the hurricane, on September 2, 2005. At the time shown in this figure, the floodwaters have been partially drained out from the flooded basin, and they are shown at elevation + 3 feet (MSL) [or +5 feet, NAVD 88.]

Cloud cover obstructed the taking of a good image of the flooding at its peak, but this basin flooded very rapidly in the first hours of the main storm surge. The levees were massively breached and catastrophically eroded on the northeastern flank; fronting the MRGO channel and Lake Borgne. In addition, two large breaches occurred at the west end of this protected basin, fronting the IHNC. The result was that this basin flooded extremely rapidly, before the storm surge had subsided, and the resulting surge-pushed floodwaters rose to an elevation of approximately + 12 feet above mean sea level in this basin. As a result, even homes and businesses located on ground well above sea level were inundated. Of course, sites on lower ground were inundated to greater depths.

After the hurricane passed, and the storm surge had subsided, a number of “notches” were deliberately excavated through several of the levees to facilitate drainage of ponded floodwaters by simple gravity flow (as indicated by the yellow stars in Figure 6.1).

6.2 The Northeast Frontage Levees

As shown in Figures 2.9 through 2.11, the initial storm surge swelled the waters of “Lake” Borgne (which is actually a bay, as it is connected directly to the Gulf of Mexico.) As the eye of the hurricane then continued to the north, the counterclockwise swirl of the winds pushed the elevated waters of Lake Borgne to the west, against the levees along the northeast frontage of the St. Bernard protected basin. The result was catastrophic erosion of the levees along much of this frontage, and the through-passage of the floodwaters.

Figures 6.3 and 6.4 show two sections of the levees along this frontage after this event. These are aerial views taken from significant elevation, and they each show many hundreds of feet of levee section that have been catastrophically eroded. In Figure 6.3, the depression in the foundation soils induced by the settlement of the now-vanished levee, and the erosion produced by the turbulent flow across the original levee footprint, is the only sign of the former presence of a levee. In Figure 6.4, a sheetpile curtain had been driven along the centerline of the levee crest, to raise a section that had settled as an interim measure until the

final stage of fill placement could re-raise this embankment section to the final design grade. The levee embankment has eroded completely from both sides of these sheetpiles, and the large diameter pipe in this figure was resting on the crest and slopes of the now vanished levee and so serves as a visual template to show the size and shape (the outline) of the levee section that is now gone.

Figure 6.5 shows another view of massive erosion along a long stretch of levees along this “MRGO frontage” section, this time a bit farther to the south (nearer to the second navigational lock structure at bayou Dupres.) Here the massive erosion is not as complete, and portions of the levee embankment remain. In this photo, the eroded detritus can be clearly seen to be strewn back behind the partially eroded levees, and the sandy (and shell sand) nature of some of this eroded material is evident.

Figure 6.6 shows a ground level view of the sheetpiles from Figure 6.4. In this photo it can be clearly seen that the sheetpiles, which had originally been driven to constant grade, have settled differentially under the pounding of the storm surge and storm driven waves. This would suggest that the cyclic wave loading may have caused pore pressure increases in the fine, sandy foundation soils into which the sheetpiles were embedded, and that this (full or partial) liquefaction reduced the bearing strength and stiffness of these foundation soils and led to the observed differential sheetpile settlements as the sheetpiles were only lightly self-loaded with regard to vertical bearing and settlements.

LIDAR surveys were performed by the USACE to document the elevation of the levee crest along the full 11-mile long northeast (MRGO) frontage both before and after Katrina. An example is shown in Figure 6.7, where the magenta line indicates the crest elevation prior to Katrina, and the darker blue line indicates the crest elevation afterwards. The photo at the top of this figure is a vertical (plan view) photographic image along the same section. The two LIDAR surveys serve to show the amount of erosion-induced crest loss along this section, and this can be correlated with the same locations in the photo at the top. Note the light material streamed back behind the levees (on the “protected side”) in the corresponding photo; representing eroded material from the levees strewn back into the inboard side swamps.

Figure 6.7 includes the large (gated) reinforced concrete navigation control structure at Bayou Bienvenue. A large barge was deposited on the crest of the levee immediately to the north of this lock structure, and this can be clearly seen in Figure 6.7. Figure 6.8 shows a second view, of the massive breach eroded at the contact between the lock structure and the adjacent levee embankment.

Figure 6.9(a) is an oblique aerial of the smaller Bayou Dupres concrete navigation structure situated farther to the south along this same MRGO levee frontage, showing a similar massive eroded breach at the juncture between the northwest end of the concrete structure and the adjoining earthen levee section. Figure 6.9(b) shows a second view, taken from the eroded breach and looking to the inboard (protected) side along the north flank of Bayou Dupres, showing the eroded detritus strewn inland from this breach. In this figure, it can be clearly seen that large fractions of the eroded material consisted of shell sand fill. The use of lightweight shell sand fill had been called for at this interface section in order to

minimize differential settlements between the embankment section and the adjacent concrete lock structure. By minimizing these differential settlements, the formation of a small settlement-induced gap between the levee and the lock structure would be prevented. As a result of using the dangerously erodeable lightweight shell sand fill, however, a massive eroded breach occurred instead.

The crest heights of the levees along much of this MRGO frontage section were several feet below design grade at the time of Katrina's arrival. This levee frontage was being constructed in stages, to allow time for settlement of the evolving levees and for dissipation of pore pressures (which results in progressive strength and stiffness gain in both the levee fill and in the underlying foundation soils, so that the softer foundation soils can safely support the increasing levee section height and weight of the next stage.) The USACE had reportedly long requested appropriation of the funds necessary to place the final stage of fill and bring this critical 11-mile long section up to full design grade. That funding did not arrive in time.

The levees along this frontage were unusually vulnerable to erosion as they were "sand core" levees, constructed largely using material available from the adjacent MRGO channel excavation. Given the nature of the local soils at this location, much of that excavated material consisted of sands and lightweight shell sands. These materials have a low intrinsic resistance to erosion (see Chapters 9 and 10), and this led to a hazardous condition. It is possible that the final fill stage, if it had arrived in time, might have provided a covering veneer of compacted clay fill (with a higher resistance to erosion), but such a covering was not in place. In addition, given the ferocity of the surge and storm waves that struck long sections along this alignment; it is not clear that a relatively thin veneer of compacted clay would have been sufficient to help very much.

As shown in the map of Figure 6.1, this levee frontage is one of only two locations where the levees protecting the three main protected basins of New Orleans are exposed directly to storm waves crossing a large body of Gulf waters (Lake Borgne) without the protection of significant swamp grounds on their outboard sides. The swamp grounds (and cypress trees) serve to damp the energy of the storm waves, reducing their height and velocity, and thus their erosive potential. It was unfortunate that this section that was so exposed to severe (unprotected) storm waves was also not yet up to full design grade, and that large portions were comprised of highly erodeable sand and lightweight shell sand fill.

It should be noted that the only other section of levee protecting one of the three main basins of New Orleans that was also exposed to open water storm waves (without significant outboard side swamp and cypress protection) is the "sister" section to the north; at the southeast corner of the New Orleans East protected basin (facing south, fronting Lake Borgne.) As discussed in Chapter 7, that "sister" section was also constructed using dredge spoils from the excavation of an adjacent shipping channel (the GIWW channel in that case), and was also comprised largely of highly erodeable sands and shell sands. That section, too, eroded catastrophically and represented the largest source of the floodwaters that catastrophically flooded the New Orleans East protected basin.

The exact nature of the erosion and breaching that occurred along this frontage section has not yet been fully agreed upon by the various investigation teams. It is the view of our

investigation that sections of this levee frontage appear to have eroded and begun to be breached prior to the storm surge reaching its full height (of approximately +16 to +19 feet, MSL) by as early as about 5:30 to 6:00 a.m.

Figure 6.10 shows the calculated hydrograph developed by IPET at this location, showing storm surge rise vs. time at this location as estimated by IPET (IPET, Second Interim Report, April 2006.) “Storm surge” is the mean water level between storm waves and troughs, so the additional height of waves, plus “run-up” as waves arrive at the levees must be added to determine when and to what extent the waters overtopped the levees. This is further complicated by the significant variations in crest elevation along this not yet completed levee frontage. The analytical prediction of Figure 6.10 matches well with the similar numerical hydrodynamic modeling performed by Team Louisiana (Kemp and Mashriqui, 2006), and both models are fairly well calibrated against regional observations of water elevations at numerous locations. The two investigation teams (IPET and Team Louisiana) differ significantly, however, in their calculated wave heights and frequencies along this MRGO frontage. IPET have calculated longer period storm waves typical of more “open ocean” conditions, and Team Louisiana have calculated shorter period waves constrained by lack of depth within the Lake Borgne embayment.

Figure 6.11 shows a schematic illustration of two different sets of erosion mechanisms for the levees along this frontage. Figure 6.11(a) shows simple “sheet flow” overtopping. This is a common mode of concern for many river levees, and also for many earth dams. In this mode, as the water flows over the top and then flows like a sheet down the rear-side slope of the levee embankment, the velocity of flow down the rear slope face accelerates and the shear stresses (erosive forces) induced by the flow increase with this increased velocity. Accordingly, erosion is initially most pronounced low on the back slope face (where the flow velocities become highest), and the embankment is eroded from the back side until the crest is breached (whereupon rapid flow through the crest rapidly enlarges the original breach.) This is the mechanism that is the customary principal design focus for the flood control levees in this region; excepting the large rivers such as the Mississippi River where scour produced by longitudinal flow of the river current itself is also a major concern.

Figure 6.11(b) illustrates two additional potential sets of erosion modes likely to have been active along sections of the MRGO frontage levees. One is the attacking of the outboard side (water side) face of the levee by storm waves. These high energy waves can scallop and erode the outboard face. They can also rush up the face toward the crest, and can erode “notches” in the crest from the front side. Subsequent waves can then pass through these notches, especially as the storm surge continues to rise, and the flow can widen the notches and also erode the back face levee slope (as discussed above as “sheet flow overtopping erosion”.) This exploitation and widening of crest notches is called crenellation, after the crenellation (notched shape) that often tops castle walls.

Figure 6.11(b) also illustrates seepage flow passing through the embankment section, and then eroding soil as it exits through the lower portion of the back side slope face. This “through flow” can cause significant erosion if the embankment soils are pervious, as was the case along significant portions of the MRGO frontage levees. As this type of erosion occurs primarily in the lower back slope face region that is also most prone to erosion by sheetflow

overtopping, it can be difficult to separate field evidence of these two types of erosion as to cause.

The highly erodeable (and pervious) sands and shell sands that comprised significant sections of the levees along this frontage were vulnerable to all three types of erosion, and would have been expected to have been damaged by waves and by through flow from the rising storm surge well before the storm surge actually overtopped some sections. Evidence of front face scalloping erosion, and “notching” at the crest and front crest lip of levees along this MRGO frontage section are presented in Figures 6.12 and 6.13.

Methods and procedures for calculation of rates of likely erosion due to the various erosive mechanisms likely to have been operating along the critical MRGO levee frontage are not well-established, and there is little agreement within the profession as to how the erodeability of the various materials present (fill types, and fill placement and compaction states.) A number of members of the ILIT team made their own estimates of likely rates of erosion, based on their perceptions of the likely fractional content of various fill types, and the types of erodeability data presented and discussed in Chapters 9 and 10, and in Appendix I. These estimates also required judgmental assessment of through flow potential, wave runup magnitudes and velocities, numbers of wave cycles at different times (and thus different storm surge stage levels), etc.

The resulting estimates varied considerably, but all agreed that there was a high likelihood that initial breaching would have initiated well before the storm surge approached within several feet of the low points along the crests along this critical levee frontage. This appears to correlate well with the observation that massive amounts of storm surge flows filled and then pushed across the open swamplands behind the MRGO frontage levees, and then crossed over the secondary (Forty Arpent) levee and filled the populous zones to the south to elevations as high as +12 feet above mean sea level.

This is further supported by the observed behavior of the “sister” levee frontage section at the southeast edge of the New Orleans East protected basin. This section, which was also comprised in part of highly erodeable fill materials dredged from the adjacent shipping channel excavation (in that case the GIWW channel), and which also fronted Lake Borgne directly, without significant outboard side swamps or cypress to dam and suppress wave energies, was clearly breached and admitted large volumes of floodwaters well before the storm surge approached the levee crests. Timing along this “sister” section, and crest heights and storm surge heights, are better documented (see Chapter 7, Section 7.3.2) than along the MRGO frontage section, as the resultant New Orleans East flooding was definitively noted and captured on videotape by workers at the nearby Entergy power plant.

As shown in Figure 6.1, it was intended that the levees along this outer frontage would bear the brunt of the storm surge. Any overtopping flow, or even flow through localized breaches, would then have available a wide swath of undeveloped swamp land into which it could flow and pond. At the back side of this swampland a lower secondary levee (the Forty Arpent Levee) was then situated to protect the populous areas to the south. Unfortunately, the unexpectedly rapid and catastrophic erosion of this outer frontage levee allowed the storm

surge to flow virtually unimpeded across the open swampland before the storm surge had begun to subside significantly.

The Forty Arpent levee was only a “secondary” levee, with crest heights on the order of Elev. + 7.5 to + 10 feet (MSL), and it was not intended to have to face the full brunt of a largely undiminished rising storm surge. As a result, the storm surge passed easily over this secondary levee, and pushed rapidly into the populated areas of St. Bernard Parish, as described previously in Chapter 2. As it arrived rapidly, and prior to significant abatement of the storm surge, the floodwaters ponded to an unexpectedly high elevation of approximately +12 feet above mean sea level. Homes and businesses on “high ground” (at elevations several feet and more above sea level) were thus unexpectedly flooded, and the depth of flooding in lower-lying areas was especially severe. The massive intruding floodwaters also had large lateral force, and pushed homes aside from their foundations (as shown previously in Figure 2.19), tossed cars like toys (see Figure 6.15), deposited large fishing boats in residential neighborhoods (Figure 6.16), and left large branches of trees on the roofs of numerous homes (e.g.: Figure 6.17).

Interestingly, the smaller (secondary) Forty Arpent levee was severely overtopped along much of its length, but it suffered relatively little erosional damage as a result. This appears to be because it was constructed of significantly better materials than the outer (MRGO frontage) levees; the Forty Arpent levee appears to have been constructed primarily of clay, with good intrinsic resistance to erosion. Figure 6.14 shows a section of the Forty Arpent levee that was apparently significantly overtopped, but which suffered only slight “cosmetic” erosional damage as a result.

The use of highly erodeable sand and shell sand fill was unfortunate along the exposed MRGO frontage levee section, and the consequences were severe. Damage to the populated areas of St. Bernard Parish was catastrophic, and the floodwaters from this populous area next began to make their way westwards towards what was now the already doomed Lower Ninth Ward.

6.3 The Two large Breaches on the East Bank of the IHNC at the Lower Ninth Ward

As the storm surge from Lake Borgne pushed westward along the east-west trending channel of the GIWW/MRGO that separates the St. Bernard and New Orleans East protected basins, it raised the water levels in the IHNC and produced two massive breaches on the east bank of the IHNC (at the western edge of the Lower Ninth Ward). These two breaches occurred at approximately 7:30 to 7:45 a.m., at an IHNC water level of approximately Elev. + 14 to +14.5 feet (MSL), as shown in Figure 6.18 (which shows a hydrograph of measured water levels vs. time in the IHNC channel.)

6.3.1 The IHNC East Bank (South) Breach at the Lower Ninth Ward

The larger of these two breaches was the south breach, and this is shown in Figure 6.19 (which is a repeat of Figure 2.13). This was a very long breach, nearly 900 feet in length, and the intruding waters entered the adjacent community with great force. As shown

in Figure 6.17, homes for several blocks were ripped from their foundations and scattered, usually in splinters, eastward across the inboard neighborhood.

Figure 6.19 also shows the sheetpile curtain that had supported the floodwall at the crest of the earthen levee at this section. It is interesting to note that the sheetpiles (which were cold-rolled steel sections) remained interlocked throughout the cataclysmic failure and the ensuing hydrodynamic loading of the massive intrushing floodwaters. The concrete floodwall is largely absent from the tops of these sheetpiles, as the sheetpiles have been stretched out (like an accordion), flattening their bent flanges in order to accommodate the extension imposed on them by the intrushing flow.

Figure 6.19 also shows a large steel barge that passed inward through this section, and came to rest near the southern end of the breach. This raised the question as to which came first; the barge or the breach?

Figure 6.20 shows the large barge, in its final resting position (prior to being cut apart with torches to remove it) atop a small yellow bus. This was not the initial resting location of this barge immediately after hurricane Katrina, however. Initially, after Katrina, the barge had come to rest a bit farther to the east. It was then re-floated several weeks later when the temporary breach repair failed during the second hurricane surge produced by hurricane Rita on September 24, 2005 (see Chapter 11), and came to rest at its current position at that time. The small yellow school bus also arrived between hurricanes Katrina and Rita, having been appropriated and used for interim transport and then abandoned in its location as shown.

There is a single large dent low on the side of the barge just around the left side of the bow (not quite visible in Figure 6.20), and a pronounced scrape on the bottom of the barge at that same location. Most of the concrete floodwall was failed in extension and flexure, with its reinforcing steel (rebar) fairly extended. There was one single section of wall which clearly evinced a major impact, however, and that was at the extreme southern end of the breach. Figure 6.21 shows a close-up view of the floodwall at this location. The rebar is compressed and bent, and the concrete crushed at this location. It was the consensus view of our investigation team that the barge had scraped along the wall and then impacted the end of the wall at this location.

As this was the extreme southern end of the very long breach; this impact was not the cause of the breach and failure. Instead, the barge was apparently traveling southwards along the IHNC (driven by the prevailing storm winds at that time) and was drawn into the breach by the inflowing waters. The barge did not enter cleanly into the breach, but struck at the south end before passing in.

That does not mean that the barge might not have struck the floodwall twice (or more times) before finally impacting the southern end of the breach, but our investigation's view is that there are other modes of failure that would have been expected to fail this section without any need for help from the barge, so that the likelihood is that the barge slipped its moorings and was eventually drawn in through a breach that was already well developed.

Figure 6.22 shows the trench that was eroded by water that passed over the top of the concrete floodwall at the south end of the large breach. (The barge can be seen at the right in this photo.) Overtopping and scour occurred at both ends of this breach feature, and the resulting scoured trenches reached depths of up to 5.5 feet in sections that did not subsequently fail. It is, of course, not possible to determine whether deeper scouring/trenching might have occurred at the actual breach inception location, as the embankment and foundation soils at the center of the breach were deeply scoured out by the massive flows in through the breach. One of the potential failure modes evaluated by our (ILIT) studies was the possibility that this scour had sufficiently laterally unbraced the concrete floodwall (and its supporting sheetpile curtain) that the lateral force of the elevated canal water was able to displace it laterals and foment a resulting breach.

Figure 6.23 shows our ILIT re-interpretation of the original boring data along this section of the east bank of the IHNC, with the locations of the two large breaches indicated. The boring data was far too sparse along this section for the importance of the design (the inboard population and properties being protected) and for the complexity of the local geology. In addition, widely spaced borings along the approximate levee centerline do not provide an adequate basis for development of appropriate cross-sections for analysis and design. An effort was made to perform pairs of borings (one roughly at the crest and another at the inboard toe) at selected locations so that cross-sections could at least be attempted, but this was still an inadequately sparse investigation. The foundation investigation for the design of these levees and floodwalls was inadequate for a project of this scope and importance, and the minor savings in drilling, sampling and testing are now dwarfed by the massive costs of the failures that resulted; both property damages and loss of life.

Figure 6.24 shows the cross-section used for our analyses of this south breach. The two pre-Katrina ("initial design") borings, Borings B-4 and B-4T, were supplemented by three additional CPT probes performed by the IPET investigation (IHBR-6.05C, 5.05C and 16.05C), and two additional borings and a CPTU probe that were performed by our ILIT investigation (Borings IHNC-S-BOR-1 and CON-1, and CPTU IHNC-S-CPT-1). The cross-section of Figure 6.24 shows the tragic failure to extend the sheetpile curtain to sufficient depth as to cut off underseepage flow through the laterally pervious "marsh" deposits at this site.

The upper embankment fill is a moderately compacted imported clay, which is underlain by an older "fat clay" (CH) fill apparently comprised of locally available lacustrine clays. The upper foundation soils are then dominated by thick deposits of high plasticity clays (CH), punctuated by two layers of marsh deposits, and there is a relatively thin but continuous stratum of low plasticity silt (ML) underlying the lower marsh unit.

Subsequent to the completion of the levee embankment and floodwall, additional sandy fill was placed on the outboard (water) side of the levee to raise the ground surface slightly above mean canal water level. Some buildings and facilities had been constructed on this made ground, but these had been removed prior to hurricane Katrina.

Figure 6.25 shows plots of data regarding strength properties vs. depth for the soils from the silt layer down (from Elevations -19 to -50 feet, MSL) beneath (a) the levee crest,

and (b) at the inboard toe of the levee (under far lesser embankment overburden). The detailed procedures and relationships used to process the CPTU data, and then to overlay the additional UUTX data to develop these plots, are presented and discussed in detail in Chapter 8, and this will not be repeated here. The lower unit of lacustrine clay clearly shows two overconsolidation “crusts” as a result of surface desiccation during early “stands” in the accretion of these deposits, and they are more normally consolidated at greater depth. These clays, in the end, do not appear to have participated in the failure that occurred.

Similarly, the relatively thin silt stratum (ML) also shows evidence of overconsolidation, and this gives it sufficient strength that it too is uninvolved the failure.

Figure 6.26 shows similar plots regarding strength properties of the far more critical upper foundation soil strata between elevations of approximately +0 to -20 feet (MSL). These deposits, consisting of interlayered marsh and clay units, are the critical soils at this site.

As described in detail in Chapter 8, a number of different approaches were taken to the processing of the available field and laboratory test data in order to evaluate and characterize these soils. Based on the CPTU measurements within the marsh deposits (both at this site, and at the 17th Street canal breach site) values of B_q were developed, and then based on the relationships of Lunne et al. (1994) and Karlsrud et al. (1996), a value of $N_{kt} = 15$ was selected for transposing the CPTU tip resistance values to the estimates of undrained shear strength that are plotted in Figure 6.26. The resulting values were then converted to values of S_u/P as shown in the far right figure of Figure 6.26, and these appear to infer three desiccation-induced overconsolidation profiles corresponding to surface exposure at three times during the evolution of these deposits. The relationship of Mayne and Mitchell (1988) was then used, again as described in Chapter 8, to cross-check the resulting relationship between S_u/P vs. OCR as a function of Plasticity Index (PI, %) for these deposits using the available UUTX laboratory test data. These were found to be consistent. Finally, the limited available in situ vane shear test data, and the UUTX laboratory test data, was co-plotted with the CPTU-based strengths, and these too were judged to be consistent (with allowances for sample disturbance and vane insertion disturbance in these soils of variable fibrous organic content).

Similar processing resulted in selection of a value of $N_{kt} = 15$ for processing of the CPTU data for the silty clay (CH/CL) stratum lying between the two “marsh” deposits. This differs from the value of $N_{kt} = 12$ that was used to process the CPTU data for the deeper layer of gray lacustrine clay of high plasticity, and it reflects the lower plasticity of this upper clay unit. Once again, the limited available in situ vane shear test data and UUTX laboratory test data were then co-plotted with the strengths as interpreted by the CPTU, and were found to be consistent (as shown).

Figure 6.27 shows the geometry and principal input parameters used to model and analyze this section using the finite element analysis program PLAXIS (2004). The “soft soil” constitutive model within PLAXIS was used to model all of the uppermost soil strata, so that both undrained and partially drained conditions could be studied within an effective stress framework. Shear strengths from Figures 6.25 and 6.26 were reduced by 15% in the marsh strata, and by 20% in the clay strata, to account for differences between the field (in situ) test

conditions and the laboratory test conditions, and the direct simple shear (DSS) conditions expected to dominate the critical field performance behavior in these analyses.

Initial analyses were performed to model the incremental construction of the levee embankments in order to establish the initial stress conditions for the subsequent analyses of the overall section performance and stability during hurricane Katrina's storm surge loading. Figure 6.28 shows the deformed mesh at the end of staged construction and consolidation under the levee embankment loads. Overconsolidation stress profiles beneath the crest, and beneath the inboard levee toe, well matched those from the available field data, and the consolidation properties were iterated slightly until the final (post-consolidation) settled profile matched well with the observed field configuration.

Analyses were then performed in which the water level within the canal was progressively raised. Transmission of pore pressures beneath the wall (and beneath the sheetpiles) was very rapid, and nearly "steady state" pore pressure conditions developed very rapidly beneath the inboard side of the levee after each increase in water levels as the lateral transmissivity of the marsh deposits was high, and the system was initially well saturated. The rate of water level rise (and subsequent decline) in the canal was based on the hydrograph of Figure 6.18.

Figure 6.30 shows conditions calculated just as the canal water level reached the top of the concrete floodwall. Plotted in this figure (as color contours) are levels of relative shear strain (shear strain developed, divided by shear strain to failure) within the levee embankment and foundation soils. As shown clearly in this figure, two distinct failure mechanisms are beginning to develop. The lower one is a shear surface concentrated at the interface between the base of the upper gray clay (CH/CL) layer and the underlying layer of marsh deposits, and the upper failure surface attempting to develop is concentrated at the interface between the top of the upper marsh stratum and the lower levee embankment fill section. Both of these mechanisms represent the results of underseepage-induced increases in pore pressures being "trapped" at the bases of less pervious overlying strata. These pore pressure increases are decreasing the strength and stiffness of the soils at these two critical interfaces.

At the water stage shown in Figure 6.30, a gap has begun to form at the outboard side of the floodwall and its supporting sheetpile curtain. When effective tensile stress was calculated between the floodwall/sheetpile wall and the adjacent soils, the analysis was temporarily stopped, the tension was eliminated by changing the mesh details to insert a small gap (and to insert hydrostatic water pressures within the gap), and the analysis was resumed. This was done iteratively, as water levels continued to rise, so that the progressive development of a water-filled gap between the floodwall/sheetpile curtain and the outboard section of the levee embankment could be modeled. At this section, within reasonable parameter variations modeled, gap formation generally initiated at canal water levels on the order of Elev. +11.5 to +13 feet (MSL), and the gap then tended to progress fairly rapidly to the base of the sheetpiles (within the next 1 to 2 feet of water level rise in the canal).

Figure 6.31 shows calculated conditions for a canal water level at Elev. +14 feet (MSL). At this stage, water is now overtopping the floodwall, the gap at the outboard side of the sheetpile wall is developed to full depth, and stability failure is occurring on the

uppermost of the two potential failure surfaces. This upper failure is serving to “protect” against further development of the lower failure surface (which can also be seen in this figure.) If the upper failure surface is strengthened a bit, to prevent the upper failure, then the lower failure becomes critical.

Figure 6.32 shows the postulated path to failure based on the finite element (PLAXIS) analyses performed. In this figure, the Factor of Safety at any given surge height was assessed by stopping the analysis at each stage of water level rise, and evaluating Factor of Safety by means of progressive $c - \phi$ reduction. Two sets of conditions were analyzed; conditions in which a “gap” was allowed to form on the outboard side of the sheetpile/floodwall (and the gap was allowed to fill with water as it opened), and a second set of analyses without allowing the opening of this gap. The light blue diamonds in Figure 6.32 represent conditions without gapping, and the yellow circles represent conditions with progressive opening of a water-filled gap.

As shown in this figure, the gap begins to open as the storm surge rises near to the top of the floodwall (at a surge elevation of about +11 to +12 feet, MSL), and the increasing lateral push of the rising surge waters finally destabilizes the system at a surge elevation of approximately +12 to +13 feet, MSL. This appears to agree closely with the observed field timing and surge levels at failure.

These analyses also include the “excavation” of a trench at the levee crest at the rear side of the floodwall representing the results of overtopping erosion at the north and south ends of the breach. The depth of this eroded trench was taken as rapidly increasing from none to 5 feet in depth as overtopping began to pass over the top of the floodwall. Additional analyses were performed for eroded trench depths of up to 7.5 feet, but this did not significantly affect the overall results; simple erosion of a scoured trench behind the floodwall, even as deep as 7.5 feet, was not sufficient as to cause the observed failure and breaching of this levee/floodwall section. The scoured trench behind the floodwall did contribute a bit to the enhancement of lateral displacement (and resultant water-filled gapping) on the outboard side, but it does not appear to have been the principal factor at this failure and breach site.

Additional analyses were performed to further evaluate both the seepage flow vs. time, and the overall stability of this levee and floodwall section. Seepage analyses, as well as conventional Limit Equilibrium analyses (by several methods, but these agreed closely and results presented herein are for Spencer’s Method) were performed using the program package GEO-SLOPE/W.

Figures 6.33 and 6.34 show the cross-sections and meshes used for conventional limit equilibrium and coupled seepage analyses of this same breach section. As shown in Figure 6.35, the rapid lateral flow through the main marsh stratum distorts the flownet, carrying pressures and equipotential contours along as it passes beneath the embankment. Figure 6.36 shows a close-up view of calculated pore pressure contours for a storm surge elevation of +14 feet (MSL). Over a considerable area at and inboard of the levee toe the net pore pressure uplift forces are slightly greater than the weight of the relatively light soils present, representing conditions prone to potential “uplift” or “blowout” at this critical location.

Figure 6.37 shows a close-up view of hydraulic gradients at this same canal surge stage. As expected, the exit gradients calculated at the toe are slightly unstable with regard to initiation of seepage erosion and piping for the relatively lightweight soils present.

A key question in these analyses is the rate at which rises in outboard side canal water levels manifest themselves in the form of increased pore pressures beneath the inboard side of the levee embankment. That, in turn, is largely a function of the lateral permeability modeled within the marsh strata, and assumptions regarding degree of initial saturation.

It was our investigation team's observation that lateral permeability was very high within at least some of the sub-strata of these variable marsh deposits, both at the two east bank IHNC breach sites at the edge of the Lower Ninth Ward, as well as at sites along the drainage canals at the north end of the main (downtown) New Orleans protected basin. Hydraulic response at nearby boreholes was very rapid, and evidence of the occurrence of high water pressures and underseepage was noted at several locations. Investigators from the IPET team were surprised by difficulties in dewatering a very shallow excavation to recover large block samples of peaty "marsh" deposits at the 17th Street canal breach site for subsequent centrifuge testing. In addition, persistent reports of underseepage and ponding of waters along this IHNC frontage at the west edge of the Lower Ninth Ward, and contractor's significant problems with dewatering of excavations along this same frontage, all bespoke of high lateral permeability within these strata.

The values of lateral permeability used in these analyses were based on experience with similar geologic units from other regions, our own field observations, and the accumulated reports indicating high lateral permeability. A best-estimated coefficient of lateral permeability of $k_h \approx 10^{-2}$ cm/sec was modeled for the most open of the marsh sub-strata, and parametric sensitivity analyses were performed for values of k_h that were five times higher, and values that were an order of magnitude (factor of 10) lower.

Figures 6.38 and 6.39 show results of these sensitivity analyses. Transient flow analyses were performed in which canal water levels were raised progressively, beginning with assumed fully equilibrated ("steady state") conditions with a canal water elevation of about +5 feet (MSL) at ~11:00 p.m. on the night of August 28th (after many hours of relatively slow surge rise to that level), then rising progressively to elevation +9 feet (MSL) by about 3:30 a.m. on the morning of August 29th, and then rising a bit more rapidly to elevation +14.4 feet (MSL) by about 8:30 a.m. (It should be noted that the failure and breach occurred at about 7:45 a.m., but that these transient flow analyses were carried forward to at least 9:00 a.m. to more fully examine progressive flow and pore pressure development.)

Figure 6.38 shows calculated pore pressures vs. time at location 1, at the top of the lower marsh stratum, directly below arrow "D" near the inboard toe of Figures 6.35 through 6.37. The horizontal light blue line at the top of this figure represents the "steady state" conditions that would eventually develop for a canal water level rise to Elevation +14.4 feet (MSL) if infinite time were allowed for full equilibration and development of steady state flow. The lower diamonds represent calculated transient pore pressures at Location 1 for the best-estimated lateral permeability of the marsh deposits, and for the upper and lower bound permeabilities. As shown in this figure, the variation in permeability does not exert a major

influence on the pore pressures, given the relatively slow rate of canal water level rise, and pore pressures within the main marsh deposit at the base of the inboard levee toe are on the order of about 85% to 92% of full “steady state” pressures at the apparent time of failure (at about 7:45 a.m.)

Figure 6.39 shows similar transient flow analyses to calculate pore pressure development at various depths beneath the location of arrow “D” in Figures 6.35 through 6.37, using the best-estimated permeabilities. Again, pore pressure development is fairly rapid, and lags only moderately behind outboard side canal water level rise.

Figure 6.40 shows the calculated gradients at the top of the lower marsh stratum (the blue line) at about 7:45 a.m., based on best-estimated permeabilities, and the exit gradients at this time as well (the red line.) The toe exit gradients are marginally unstable, given the lightweight materials present, and represent conditions likely to give rise to the inception of piping erosion.

Figure 6.41 shows the progressive development of pore pressures at the top of the lower marsh stratum vs. time. As shown, there is a considerable area over which the hydraulic uplift forces progressively grow to become somewhat larger than the total overburden stresses; representing a condition that could lead to uplift and “blowout” at this location.

Finally, Figures 6.42 and 6.43 shows analyses of limit equilibrium (Spencer’s Method) for failure surfaces passing (a) along the interface at the top of the upper marsh stratum, and (b) along the interface at the top of the (lower) main marsh stratum, for a canal water elevation of +14 feet (MSL). Both sections are marginally unstable at this condition with regard to lateral translation of the inboard portion of the levee embankment, pushed sideways by the outboard side canal water pressures (including a water-filled gap at the outboard side of the sheetpiles), and in both cases the foundation soil strengths have been critically reduced by underseepage-induced pore pressure increases.

Figures 6.42 and 6.43 likely overestimate the overall lateral translational stability at this stage of canal water level rise, as it is likely that piping erosion would have at least initiated at the inboard toe region by this stage, and the calculated hydraulic uplift pressures in the inboard toe region are high enough that “buckling” of the passive toe block helping to restrain the lateral translations of Figures 6.42 and 6.43 might further reduce the overall stability.

As shown by these analyses, as the canal water level rises above about + 13 to +14 feet (MSL) this section becomes analytically unstable by a number of potential mechanisms, all of them associated with underseepage flow passing beneath the sheetpile curtain. These potential mechanisms are:

1. Seepage erosion and piping due to excessive exit gradients at the inboard toe.
2. Hydraulic uplift or “blowout” at the inboard toe.

3. Translational stability failure, as a result of reduction in strength of the foundation soils at the inboard side due to underseepage-induced pore pressure increases.

Based on the length of the breach feature (approximately 900 feet), it is most likely that the mechanism that won the race to failure at this site was translational instability due to underseepage-induced pore pressure increases, and resulting strength reduction within the inboard side foundation soils. It is certainly possible, however, that all three mechanisms contributed at least in part. Figure 6.44 shows the postulated most likely path to failure, based on both the finite element and the coupled transient flow/limit equilibrium analyses. The postulated failure path proceeds up the “un-gapped” limit equilibrium path at the right of Figure 6.44 until a gap on the outboard side of the sheetpiles begins to open and fill with water at a canal water elevation of about +12 to +13 feet (MSL). The failure mechanism then transitions to the “water-filled gap” limit equilibrium case (the left-most line in Figure 6.44), and as the canal water level continues to rise the overall section becomes unstable (in underseepage-induced lateral translational foundation instability) at a canal water level of approximately +14 feet (MSL).

This contradicts the initial conclusions of the Draft Final Report by the IPET investigation (IPET; June 1, 2006), and also the initial hypotheses of the ASCE and NSF-sponsored field investigation teams; all of which favored the hypothesis that the failure and breach at this site had resulted from overtopping flow over the floodwall which eroded a trench along the back side of the wall (as shown previously in Figure 6.22), resulting in laterally unbracing the wall so that it was then pushed over by the surge water pressures on its outboard side.

Our investigation’s view is that, while overtopping and trenching were in fact occurring, it was underseepage-induced instability that actually developed the more critical mechanism that led to failure at this site.

The depth of overtopping-induced trench erosion at the north and south shoulders of the breach never reached depths greater than 4.5 to 5 feet. It might be inferred that a low spot along the crest of the floodwall occurred at the breach location, and that somewhat deeper erosional trenching resulted, but our finite element analyses show that even excavation of a trench as deep as 7 to 8 feet by overtopping erosion does not sufficiently unbrace the wall as to foment a lateral wall failure at surge heights overtopping the wall by as much as 1.5 feet. Instead, the contribution of overtopping and erosion of a trench at the inboard toe of the floodwall was more likely to have, at best, slightly accelerated the timing of this failure by adding to the propensity of the floodwall to deflect laterally slightly and thus develop a “gap” into which water could flow and then apply additional lateral pressure against the sheetpile curtain to promote the lateral translational stability of the inboard side of the levee embankment.

Our finite element analyses, performed with eroded “trenches” of various depths (from none, to as much as 8 feet) suggest that the trench erosion likely helped to exacerbate the initiation of “gapping” at the outboard side of the floodwall at a slightly lower canal water elevation than would have occurred without this erosion, but that it was not the critical contributor to this failure.

This is an important issue with regard to repair and reconstruction. The USACE has expended considerable effort and resources to replace “I-walls” with “T-walls”, and to install concrete splash pads behind additional I-wall sections, in order to prevent failures due to the mechanism of overtopping, erosion of a trench at the rear side of the I-walls, and failure due to the resulting unbracing of the wall sections. Although also useful, this will not also deal effectively with the underseepage issues that appear to have been the actual cause of failure at this site; and there appear to be unreasonably short sheetpile curtains (insufficient as to effectively cut off underseepage flows) at other locations throughout the New Orleans regional flood defense system. This is a potentially pervasive problem throughout the system, and it should be evaluated system-wide, and remedied as necessary.

The IPET Final Draft Report notes that possible modes of failure initially considered at this site included “sliding instability and piping and erosion from underseepage.” The report then goes on to say

Piping erosion from underseepage is unlikely because the I-walls were founded in a clay levee fill, a marsh layer made up of organics, clay and silt, and a clay layer. Because of the thickness, the low permeabilities of these materials, and the relatively short duration of the storm, this failure mode was considered not likely and was eliminated as a possible mode of failure.

This greatly underestimates the permeability, and especially the laterally permeability of the marsh deposits. It also continues the very dangerous assumption that underseepage was not a serious problem for “short duration” storm surge loading that plagued the original design of many sections of the New Orleans regional flood defense system, and led to use of sheetpile curtains that were far too short to effectively (and safely) cut off underseepage flows. At least four major failures (and breaches) that caused large portions of the overall flooding damage and loss of life during hurricane Katrina appear to have been principally due to lack of appreciation of underseepage, and resulting inadequate (short) sheetpile cut-offs. These are the major breach at the west bank near the north end of the London Avenue drainage canal (see Section 8.3.9), the major breach at the east bank of the London Avenue drainage canal farther to the south (see Section 8.3.8), and the two breaches on the east bank of the IHNC at the west end of the Lower Ninth Ward discussed in this current section and in Section 6.3.2. Exoneration, a priori, of underseepage dangers should be discontinued immediately, and underseepage analyses should be required for the full regional flood protection system.

Demonstration that underseepage occurred at this site can be based on arguments of analogous conditions and levee performance at this site, and at the London Avenue drainage canal breach sites, as well as at the site immediately to the north (as described in the next section.) It can also be based on the observed difficulties encountered by McElwee Construction in dewatering an excavation near the breach site immediately to the north (due to massive underseepage flow through the marsh deposits that were not adequately cut off at that site either.)

In addition, as noted in the IPET Draft Final Report in discussion of the two massive breaches at the west end of the Lower Ninth Ward:

Although it is clear that the walls were overtopped, and that their stability was compromised by the erosion that occurred, it is also clear that one of the east side breaches occurred before the wall was overtopped. Eyewitness reports indicate that the water level in the 9th ward near Florida Avenue was rising as early as 5:00 AM, when the water level in the IHNC was still below the top of the floodwall. Stability analyses indicate that foundation instability would occur before overtopping at the north breach on the east side of the IHNC. This breach location is thus the likely source of the early flooding in the 9th Ward. Stability analyses indicate that the other three breach locations would not have failed before they were overtopped.

Unfortunately, even IPET's own analyses do not suggest a high likelihood of failure of the north breach section at the canal water levels present as early as 5:00 a.m. (approximately Elev. + 9 feet, MSL), so this would not appear to be the explanation for the observed water in the neighborhood. Instead, it is proposed that the observed water rise on the inboard (protected) side near Florida Avenue was more likely the result of large underseepage flows through the highly pervious "marsh" deposits along this frontage.

Finally, clear and uncompromising evidence of the high lateral permeability of these deposits at this site is presented in Figure 6.45, which shows a well-developed classic crevasse splay that resulted from reverse underseepage through these same highly pervious marsh deposits as the ponded floodwaters drained out from the Lower Ninth Ward after the hurricane passed.

The New Orleans District of the USACE must stop "assuming" that short-term storm surges do not pose a significant risk associated with underseepage, and should instead begin assuming that such underseepage is a potential risk and that it must be addressed either: (1) with testing and analyses, (2) by means of sheetpile curtains extended deeply enough to effectively cut off potentially dangerous underseepage, or (3) by means of wider and heavier levee embankments (including inboard side stability berms) and the use of filtered drains at the inboard toe of the levee to "vent" and thus draw down the potentially dangerous pore pressures in that vicinity.

6.3.2 The IHNC East Bank (North) Breach at the Lower Ninth Ward

Figure 6.46 shows an aerial view of the partially repaired breach that occurred just to the north of the breach discussed in the preceding Section 6.3.1. This second breach feature was a much shorter feature, with a length of only approximately 250 feet.

This narrower, deep failure had similar initial geometry and stratigraphy to that of the far longer section immediately to its south, as shown by the cross-section in Figure 6.47. At this section, there is only the one main marsh layer, but most of the other soil conditions are very similar to those at the adjacent breach section to the south.

Figures 6.47 and 6.48 show the cross-section and finite element mesh used for limit equilibrium and coupled seepage analyses of this section.

Soils data at this site were sparse, and consisted of a single pre-Katrina boring (located nearby but just off-site, and with higher embankment overburden loads than at the breach site), two ILIT borings and one ILIT CPTU probe, and several additional IPET CPT probes. We were never able to fully determine the locations of the IPET CPT's in plan view, but it was assumed that they were located in adequate proximity as to be "representative", and their elevations were known with good precision so that these data could be used to at least cross-check the other data available. Cross-checking the limited data (mainly from two CPTU probes) with the data from the breach site immediately to the south showed strong compatibility; accordingly similar properties (and OCR profiles, etc.) were modeled for similar soil units at this section.

Figure 6.49 shows the calculated flownet equipotential lines for a canal water surge elevation of +14 feet (MSL). Once again, as with the larger breach just to the south, the flow travels through the continuous marsh layer that was left frustratingly open to flow by the shallow sheetpiles that were an inadequate cut-off at this site.

Figure 6.50 shows pore pressure contours for the same conditions as Figure 6.49. Once again the hydraulic uplift pressures represent potential instability with regard to lifting or "blowout" of the thin surficial strata of impervious and relatively lightweight soils overlying the marsh stratum at and near the inboard toe.

Similarly, as shown in Figure 6.51, seepage exit gradients at and near the inboard toe are massively unsafe with regard to the initiation of seepage erosion and piping in these relatively lightweight soils.

And finally, as with the adjacent breach section to the south, the section is also marginally unstable with regard to limit equilibrium (Spencer's method), as shown in Figure 6.52, as a result of underseepage-induced pore pressures and resultant loss of strength. The most critical failure surface this time passes through (and largely within) the main marsh layer, though a secondary failure surface concentrated near the interface between this marsh layer and the overlying clay layer has a nearly similar (unstable) factor of safety.

Figures 6.53 and 6.54 show calculated transient pore pressures (6.53) at the top of the marsh stratum beneath the inboard toe of the levee, and (6.54) at various depths beneath the inboard levee toe. As for the breach section immediately to the south, the upper and lower bound lateral permeability estimates are also shown in Figure 6.53; and again a large fraction of the overall rise in canal water levels has resulted in corollary water pressure increases at the inboard toe region by about 7:00 to 8:00 a.m.

Figure 6.55 shows the progressive increase in pore pressure at the top of the marsh stratum vs. time, and the pore pressures are high enough to pose a very high risk of hydraulic uplift (or "blowout") at the inboard toe region.

Figure 6.56 shows a potential path to failure by means of lateral translational foundation instability; reaching a condition of marginal lateral instability (with full

development of a water-filled gap at the outboard side of the sheetpile curtain) at a canal water elevation of approximately +13 to +14 feet (MSL).

Here again, as with the larger breach section immediately to the south, this breach section is analytically unstable by a number of potential mechanisms, all of them associated with underseepage flow passing beneath the sheetpile curtain. These potential mechanisms are:

1. Seepage erosion and piping due to excessive exit gradients at the inboard toe.
2. Hydraulic uplift or “blowout” at the inboard toe.
3. Translational stability failure, as a result of reduction in strength of the foundation soils at the inboard side due to underseepage-induced pore pressure increases.

As with the larger breach to the south, it is our investigation’s position that despite the fact that overtopping (and resultant erosion at the inboard toe of the floodwall) was also occurring, this failure was the result of one or more of the underseepage-induced mechanisms above. (Two or more of these may have acted in concert.)

This site has a well-documented history of underseepage problems; McElwee Construction had great difficulty dewatering an excavation at this site during earlier construction, and residents of the neighborhood had also previously reported problems with seepage at the inboard toe.

Based on the geometry of the post-failure configuration (see Figure 6.46), this narrow, deep failure appears to have most likely caused by either by seepage erosion and piping, or by a combination of hydraulic uplift (“blowout”) followed by piping. The calculated high exit gradients, and the hydraulic uplift pressures at the inboard toe region, would strongly support this.

The IPET interim draft report also concluded that foundation instability was the cause of the failure and breach at this site. The failure mechanism favored in those analyses, however, was based on a semi-rotational failure dominated by undrained shear failure through the soft clays underlying the marsh stratum, as shown in Figure 6.58. IPET concluded that this failure occurred at a relatively early stage, at a canal water level of only Elevation +9 feet (MSL), and that this early failure accounted for observations of ponding of water along this general levee frontage well in advance of the failure of the larger breach section to the south.

Figure 6.59 shows the IPET interpretation of shear strength data for this section, and the red lines are the IPET shear strength profiles for stability analyses (IPET: June 1, 2006.) In this figure, the values of undrained shear strength based on the CPT tip resistance data are based on a CPT tip factor of $N_k = 15$. This appears to be an overly conservative value of N_k within the lower clay stratum, as the CPT-based shear strengths within this stratum are significantly lower than the trend based on the unconsolidated-undrained triaxial tests on “undisturbed” samples obtained with a 5-inch diameter thin-walled fixed-piston sampler. In

any case, the shear strength profile used by the IPET analyses within this layer is well to the left (lower than) the vast majority of the data available.

Our own studies determined (based on B_q values from the CPTU) indicated that values of $N_{kt} = 12$ were more appropriate for this lower soft clay (CH) layer, and the resulting re-interpretation of this CPT data based on $N_{kt} = 12$ is shown (with a dark blue trace) superposed over the previous Figure 6.59 in Figure 6.60. Similarly, the dark blue lines in Figure 6.60 show our (ILIT) interpretation of the layering at this location, and the light blue dashed lines show our interpretations of shear strength vs. depth at this section. The IPET shear strength interpretation, in addition to being low, was also based on the assumption that this lower clay stratum was normally consolidated over its full depth. As shown previously in Figures 6.25 and 6.26, our own interpretations showed several clear desiccation-induced overconsolidation profiles near the middle and top of this clay layer, and additional moderate overconsolidation near the base (likely to the base being well-drained and thus partially overconsolidated due to secondary compression), and these are reflected in our ILIT shear strength profile. In this figure, the CPTU-based shear strengths (based on $N_{kt} = 12$) can be seen to be in better agreement with the other shear strength data, and the overall shear strength vs. depth profile is more consistent with this data.

Repetition of the limit equilibrium analysis (Spencer's Method) of the failure surface shown in Figure 6.58, but using our own (ILIT) interpretation of undrained shear strengths within the critical lower clay layer (Figure 6.60) results in a calculated factor of safety, even conservatively assuming the presence of a water-filled gap on the outboard side of the sheetpile curtain, of $FS = 1.89$. This underestimates the actual overall Factor of Safety, which should actually be on the order of 10% to 15% higher based on three-dimensional considerations for this narrow, deep failure. It is therefore not likely that a deep, semi-rotational failure occurred, early on and at a relatively low canal water level, at this site.

The need of the IPET analyses to provide an early failure at this north breach site in order to explain the significant observed ponding of waters along this frontage prior to the occurrence of the large breach farther to the south, and to do so without consideration of underseepage as a potential source of this water, resulted in an apparently unrealistic analysis and an indefensible failure mechanism.

If the IPET team had been made aware of the pervasive history of underseepage problems along this frontage, they would surely have considered and analyzed underseepage-related failure modes for the two large breaches along this section of the east bank of the IHNC. This information was apparently not available to the IPET analysis team, however, reflecting insufficient communication between groups and teams across the overly modular, sub-team-organized IPET studies. In addition, the pervasive failure of the New Orleans District of the USACE to adequately consider and analyze underseepage during pre-Katrina design of considerable portions of the regional flood protection system was continued in the post-event IPET studies of these two failed sections.

The New Orleans regional flood protection systems need to be thoroughly re-assessed, and re-analyzed as necessary, with regard to potential additional underseepage-related

vulnerabilities. And then these must be mitigated in order to develop a safe and reliable overall system..

6.3.3 Summary

The two large breaches on the east bank of the IHNC (at the west end of the Lower Ninth Ward) both occurred at sites where overtopping occurred. Despite the occurrence of overtopping, and resultant erosion of trenches at the inboard sides of the concrete floodwalls, this overtopping does not appear to have been the cause of the two failures. Instead, these two failures appear to have resulted from underseepage-induced instability; either due to erosion and piping at the inboard toe, “blowout”, or translational instability due to strength reduction in the inboard side foundation soils due to underseepage-induced pore pressure increases.

This represents a potentially critical difference from the findings to date from the Corps’ IPET study; as the remedy for overtopping, trench erosion, and unbracing at the top of the floodwalls is very different from the remedy for underseepage-induced instability problems. The USACE has invested large resources to replace “I-walls” with “T-walls”, and to install concrete splash pads behind additional “I-wall” sections. This is laudible, but it will not also effectively mitigate underseepage-related problems.

Remedies for the underseepage related problems revealed by these analyses would include either extension of the sheetpile curtains to greater depths in order to more effectively “cut off” underseepage, or widening of the levee embankments to the inboard side and installation of filtered drains at the inboard toes in order to safely draw down the underseepage-induced high pore pressures in that area.

Analyses of the IHNC failure sections, and of sections of the three drainage canals in the main (downtown) New Orleans protected basin (see Chapter 8), have shown that unreasonably short sheetpile curtains of too limited penetration as to effectively cut off underseepage are likely to be endemic throughout many parts of the New Orleans regional flood protection system. Indeed, the USACE at a number of breach repair sites is replacing sheetpiles with (pre-Katrina) lengths of 18 to 24 feet with far longer (deeper penetrating) sheetpiles with lengths of 60 feet and greater as part of the repair operations; an unusually frank admission that significantly deeper sheetpiles were warranted at those sections.

There is now a need to review, and to re-analyze as necessary, essentially the entire regional flood protection system with regard to potential vulnerability associated with underseepage (and inadequately deep sheetpiles), and to remedy these problems at sites where necessary in order to ensure overall safety of the system.

6.4 Summary and Findings

The catastrophic flooding of the St. Bernard and Lower Ninth ward protected basin was primarily due to: (1) catastrophic erosion of the MRGO frontage levees, and (2) a pair of large failures (and breaches) on the east bank of the IHNC at the west end of the Lower Ninth Ward.

The catastrophic erosion of large portions of the nearly 11-mile long MRGO frontage levees was the result in large part of the use of unsuitable sand and shell sand fills (with low resistance to erosion) for major portions of these embankments. Large portions of these fill materials came from spoils dredged from the excavation of the adjacent MRGO channel, and the short-term savings achieved by the use of these soils now pale in comparison to the massive damages and loss of life that resulted. Because these levees eroded so rapidly, and so massively, the storm surge was able to push largely undiminished across a wide area of undeveloped swampland behind the main frontage levees, cross a lower secondary levee (the Forty Arpent levee) that has not been intended to have to face an undiminished rising storm surge, and then charged into the populated zones of St. Bernard Parish with catastrophic consequences.

Because it passed so quickly and so completely through the frontage levees, the surge filled the St. Bernard basin to an elevation of +12 feet above sea level; inundating homes and businesses located well above sea level that had expected to be safe, and inundating lower lying properties to great depths.

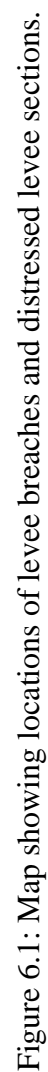
The use of intrinsically highly erodeable fills, especially clean sands, and the even more dangerous lightweight shell sands, should be reconsidered. The use of such materials as levee embankment fill, especially without taking appropriate measures to mitigate the erosional hazards associated with these (e.g.: sheetpile cutoff, erosion protection and armoring of exposed slope faces and crests, etc.) is inadvisable when constructing levees intended to protect large populations at risk.

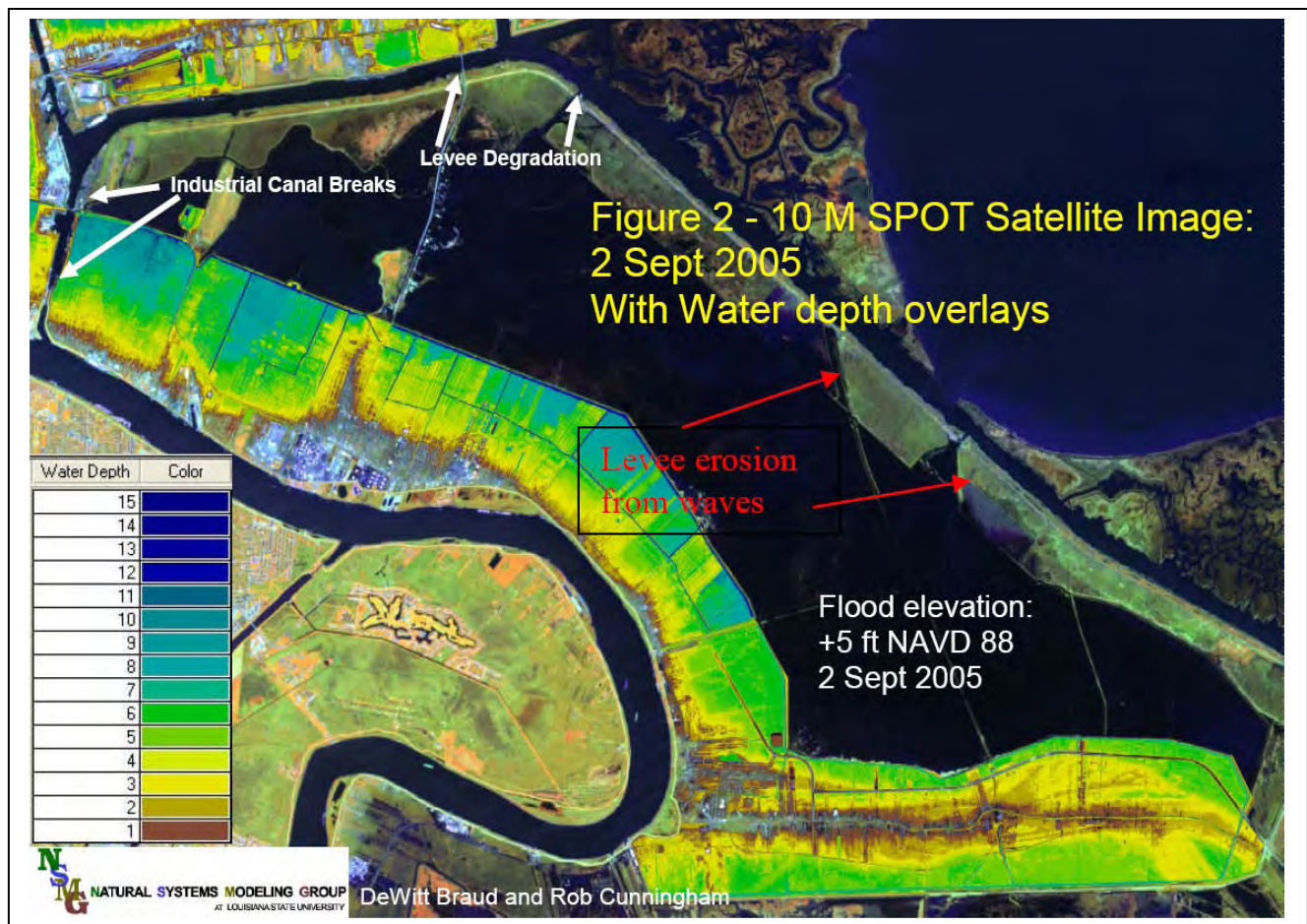
The two large breaches at the east bank of the IHNC (at the west end of the Lower Ninth Ward) both appear to have resulted not from overtopping, but rather from underseepage beneath the inadequately deep sheetpile curtains at these two sections. Overall, four of the eight most significant failures (breaches) that occurred during hurricane Katrina (the eight breaches that caused the greatest damages and loss of life) appear to have been due to inadequate attention to underseepage during initial design, and resulting sheetpile curtains that were far too short as to suitably cut-off or minimize these underseepage flows (see also Sections 8.3.8 and 8.3.9). This appears to be a widespread problem throughout the New Orleans regional flood protection system; the entire system should be re-evaluated with respect to this potential hazard, and mitigation implemented as necessary.

6.5 References

- GEO-SLOPE/W (2004) "Complete Set of Manuals", John Krahn (Edit.), Calgary, Alberta, Canada.
- IPET – Interagency Performance Evaluation Task Force (2006) *Performance Evaluation Status and Interim Results*, Report 2 of a Series, Performance Evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System, March 10, US Army Corps of Engineers.

- Karlsrud, K., Lunne, T. and Brattlien (1996) "Improved CPTU interpretations based on block samples", Nordic Geotechnical Conference, 12 Reykjavik 1996. Proc, Vol.1, pp 195-201.
- Kemp, P., (2006), Personal Communication
- Lunne, T., Lacasse, S., and Rad, N.S., (1994) "General report: SPT, CPT, PMT, and recent developments in in-situ testing", Proceedings, 12th International Conference on soil mechanics and foundation engineering, Vol.4, Rio de Janeiro, pp. 2339-2403
- Mashriqui, H., (2006), Personal Communication
- Mayne, P.W. and Mitchell, J.K. (1988) "Profiling of Overconsolidation Ratio in Clays by Field Vane", Canadian Geotechnical Journal, Vol 25, No 1, February, pp 150-157.
- PLAXIS Finite Element Code for Soil and Rock Analyses (2004). "Complete Set of Manuals", V8.2, Brinkgreve y Vermeer (Edit.), Balkema, Rotterdam, Brookfield.
- U.S. Army Corps of Engineers. (1968) *Lake Pontchartrain, LA and Vicinity Lake Pontchartrain Barrier Plan, Design Memorandum No 2, General Supplement No. 8, Inner Harbor Navigation Canal Remaining Levees*, New Orleans District, New Orleans LA.
- U.S. Army Corps of Engineers. (1969) *Lake Pontchartrain, LA and Vicinity Lake Pontchartrain Barrier Plan, Design Memorandum No 2, General Supplement No. 8, West Levee Vicinity France Road and Florida Avenue*, New Orleans District, New Orleans LA.
- U.S. Army Corps of Engineers. (1971) *Lake Pontchartrain, LA and Vicinity Lake Pontchartrain Barrier Plan, Design Memorandum No 2, General Supplement No. 8, Modification of Protective Alignment and Pertinent Design Information I.H.N.C. Remaining Levees West Levee Vicinity France Road and Florida Avenue Containerization Complex*, New Orleans District, New Orleans LA.





Source: LSU Hurricane Center

Figure 6.2: Depth of flooding of St. Bernard Parish and the Lower Ninth Ward on Sept. 2nd (4 days after Hurricane Katrina).



Figure 6.3: Catastrophically eroded levee section along the northeast frontage of the St. Bernard Parish protected basis, fronting the MRGO channel.



Figure 6.4: Catastrophic erosion of levee embankment leaving central sheetpile curtain; also along the northeast frontage of the St. Bernard Parish protected basin fronting the MRGO channel.



Figure 6.5: Extensive erosion of levees along the MRGO frontage at the northeast edge of the St. Bernard Parish protected area.



Figure 6.6: View of sheetpiles left behind at catastrophically eroded section of the MRGO frontage levee.

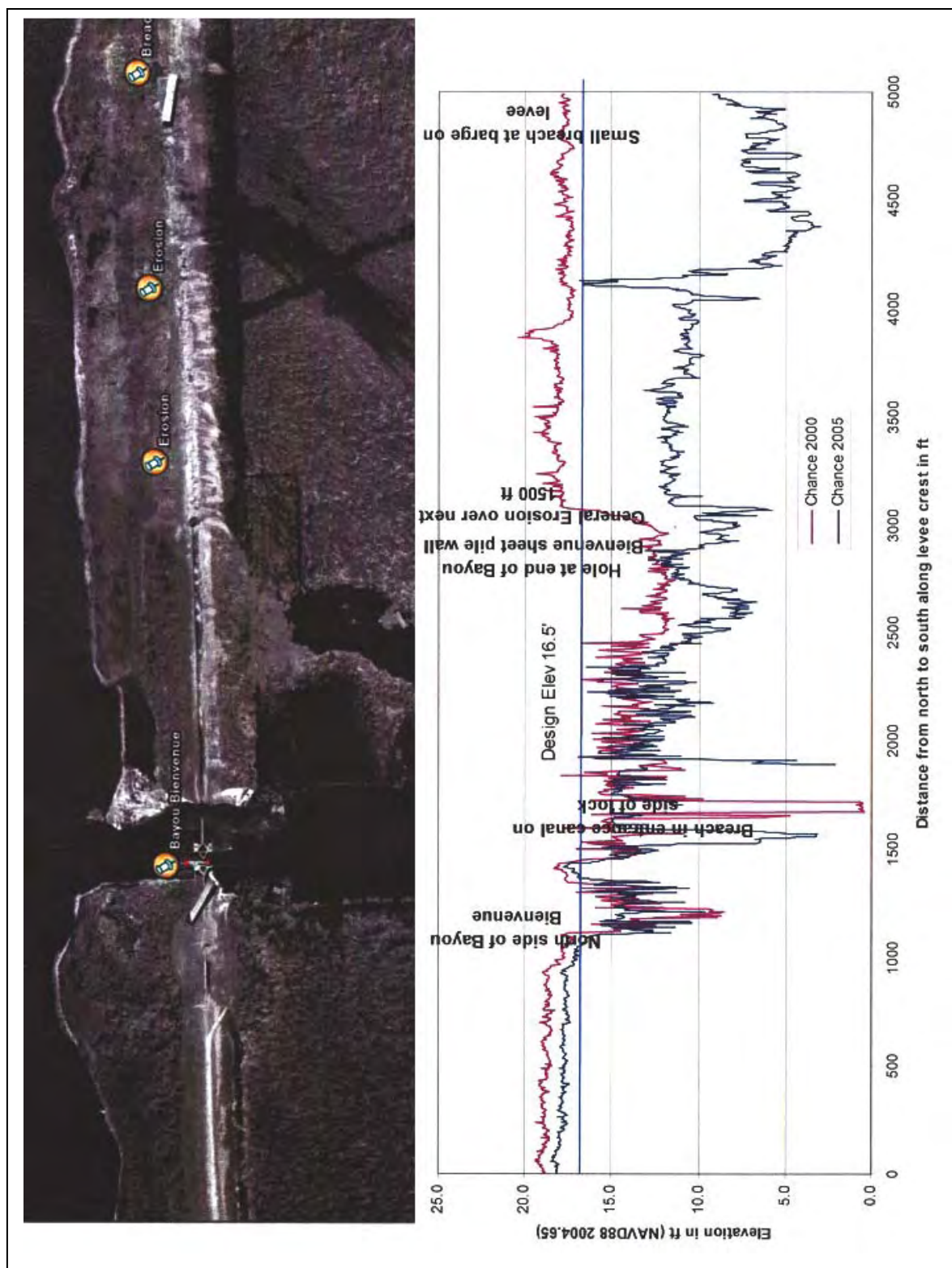


Figure 6.7: LIDAR survey data showing crest elevations before and after hurricane Katrina along a section of the MRGO frontage levees at bayou Bienvenue, and corresponding post-event plan view aerial photo along the same section.



Figure 6.8: Large eroded breach at the contact between the south end of the massive concrete navigational lock structure at Bayou Bienvenue and the adjacent levee section.



Photo courtesy of the U.S. Army Corps of Engineers

Figure 6.9(a): Aerial view of the large erosional breach at the contact between the north end of the concrete navigational lock structure and the adjoining levee embankment at Bayou Dupres.



Figure 6.9(b): View looking to the inboard side from the breach shown in Figure 6.5 above; showing eroded shell sand detritus deposited from the breach.

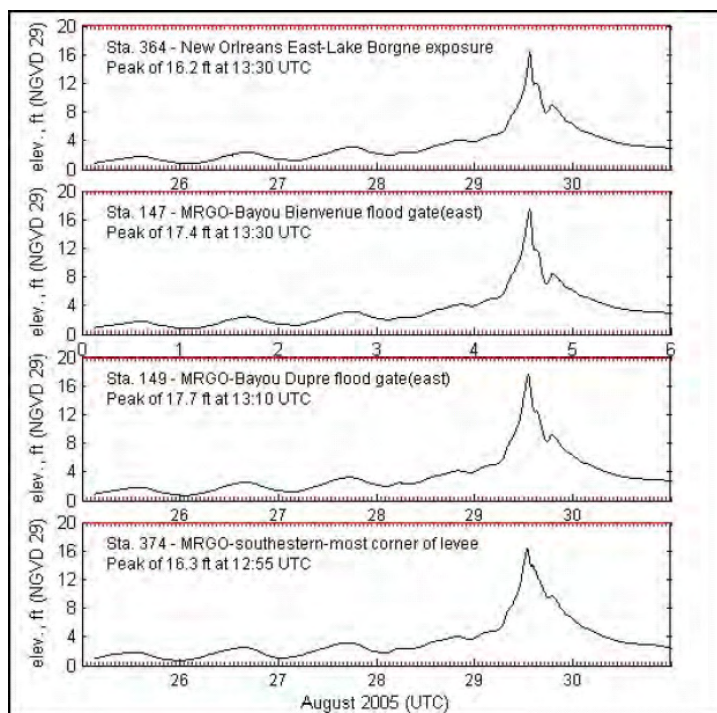
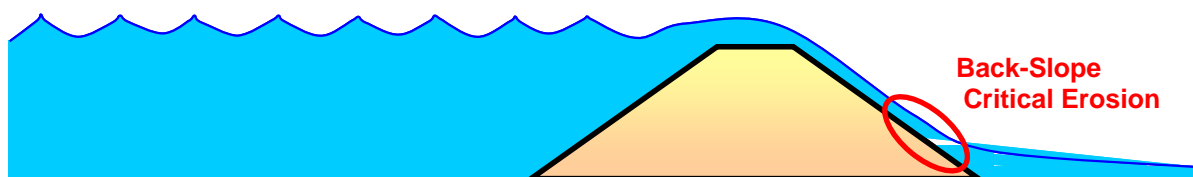
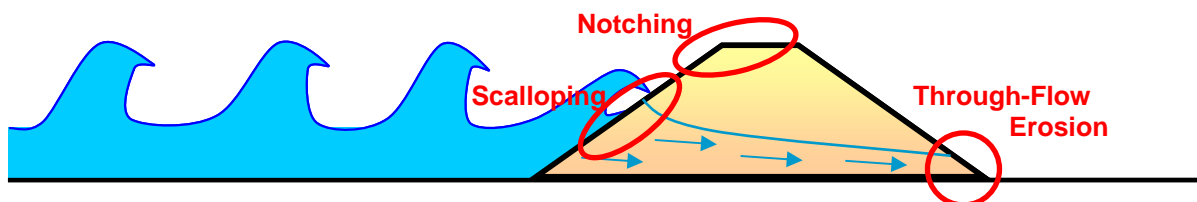


Figure 6.10: Approximate hydrograph of storm surge elevation (feet, MSL) vs. time at the west end of Lake Borgne. [IPET Interim Report, April, 2006]



(a) Sheet flow overtopping erosion of the lower back slope face



(b) Wave erosion of the front face, and through-flow erosion of the lower back face

Figure 6.11: Schematic illustration of two different sets of modes of levee erosion.



Figure 6.12: Photo of outboard side wave-induced erosion on the MRGO levee frontage at the northeast edge of the St. Bernard/Ninth Ward protected area.



Figure 6.13: Photo of outboard side levee erosion and crest “notching”, as well as crenellation, due to storm wave erosion and overtopping along the MRGO frontage levees at the northeast edge of the St. Bernard/Ninth Ward protected area.



Figure 6.14: View of the secondary (Forty Arpent) levee across the middle of the St. Bernard protected basin.



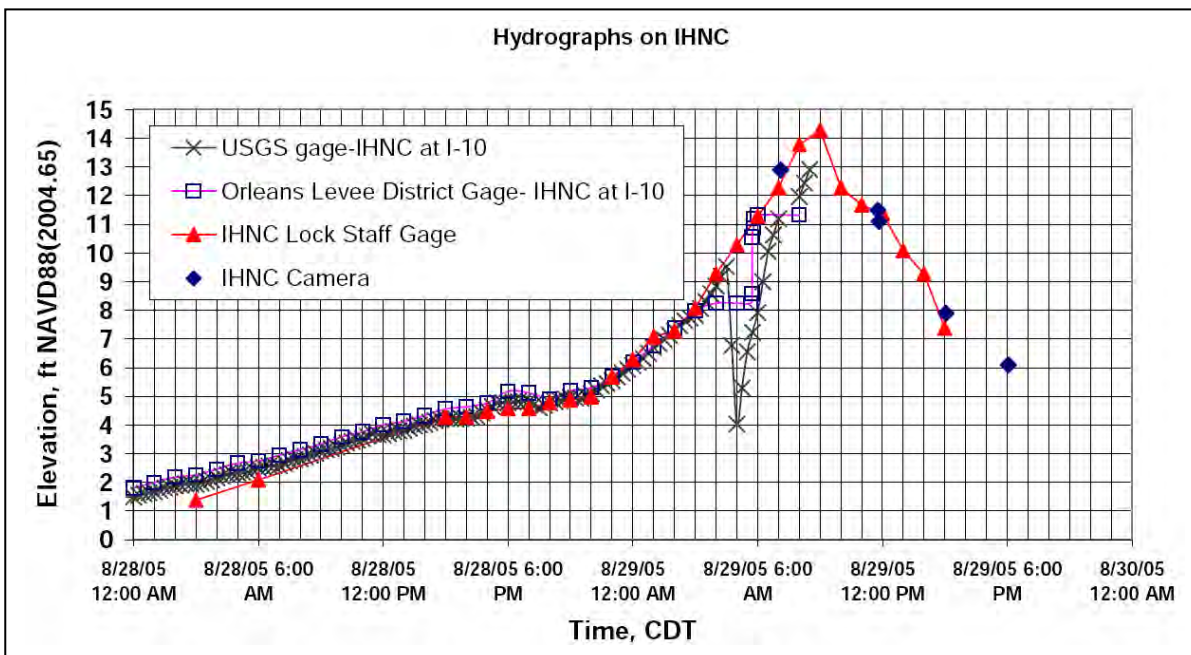
Figure 6.15: View of tree limbs and detritus on roofs of homes in St. Bernard Parish.



Figure 6.16: Car tossed and flipped in St. Bernard Parish.



Figure 6.17: Boat deposited in neighborhood in St. Bernard Parish.



Source: IPET Interim Report No. 2; April, 2006

Figure 6.18: Hydrographs showing measured (and photographed) water levels at gage stations along the Inner Harbor Navigation Channel (IHNC).



Figure 6.19: Oblique view of the (south) levee break at the Inner Harbor Navigation Canal into the lower Ninth Ward.



Photograph by: Rune Storesund

Figure 6.20: Close-up view of the large barge that entered through the south breach at the east bank of the IHNC at the west end of the Lower Ninth Ward.



Photograph by: Rune Storesund

Figure 6.21: Close-up view of crushed (impacted) concrete floodwall at the south end of the south breach at the east bank of the IHNC at the west end of the Lower Ninth Ward.



Photograph by: Rune Storesund

Figure 6.22: Eroded trench at the rear (inboard) side of the floodwall at the south end of the south breach at the east bank of the IHNC at the west end of the Lower Ninth Ward.

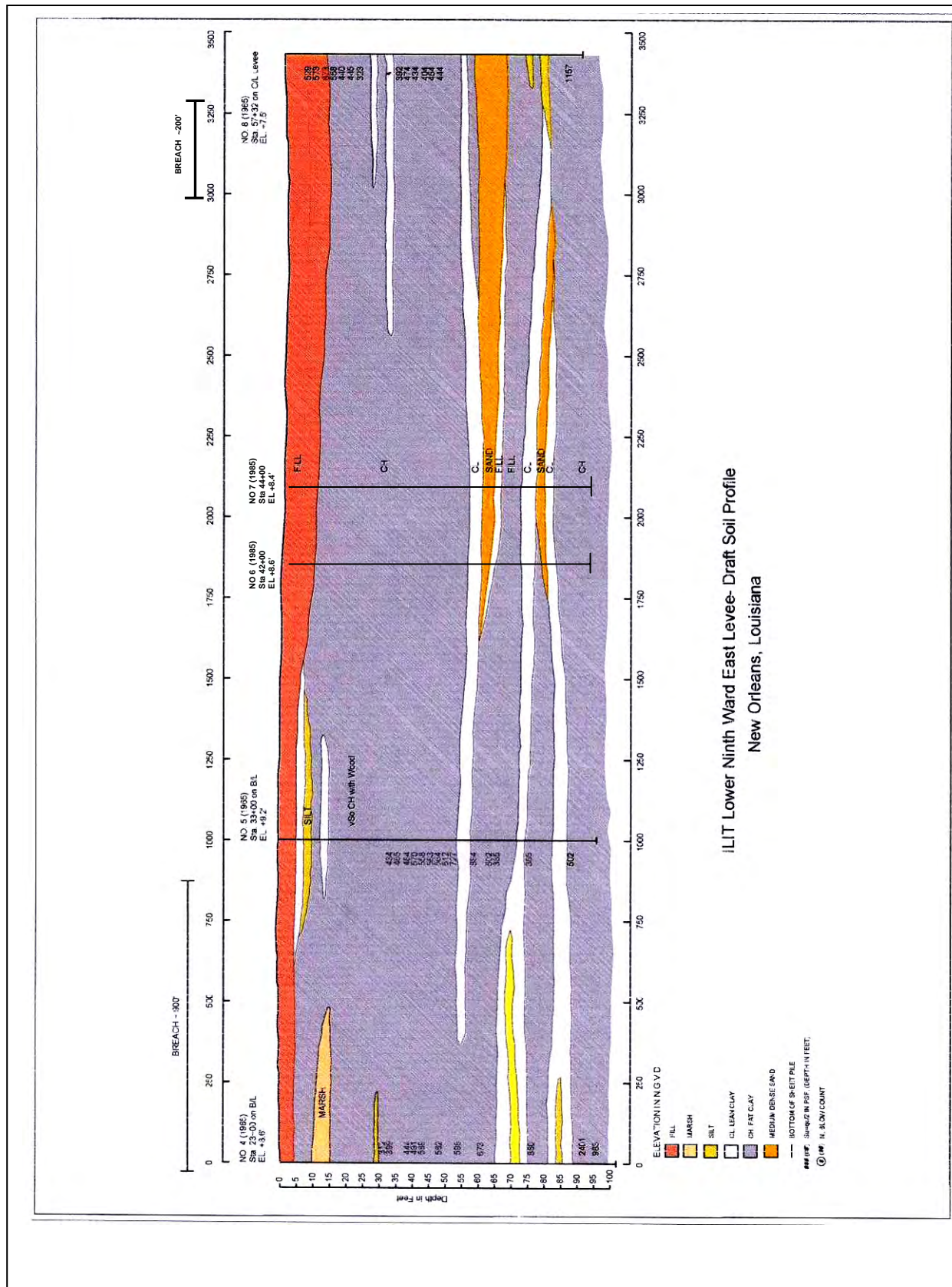


Figure 6.23: Re-interpreted longitudinal subsurface soil profile, showing location of breach section on the east bank of the Lower ninth Ward Levee.

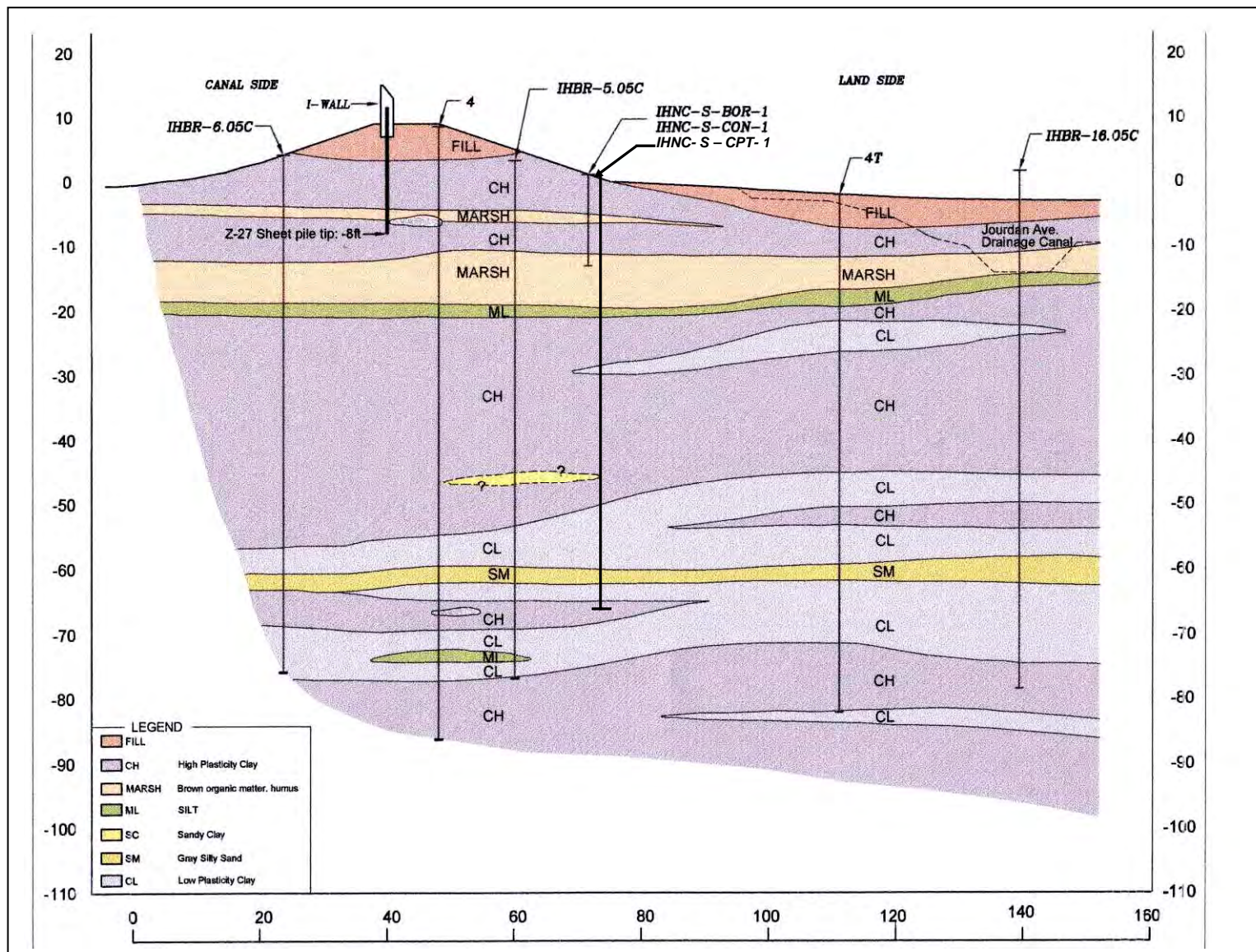


Figure 6.24: Cross-section showing location of borings for Lower Ninth Ward, East Bank, South Breach.

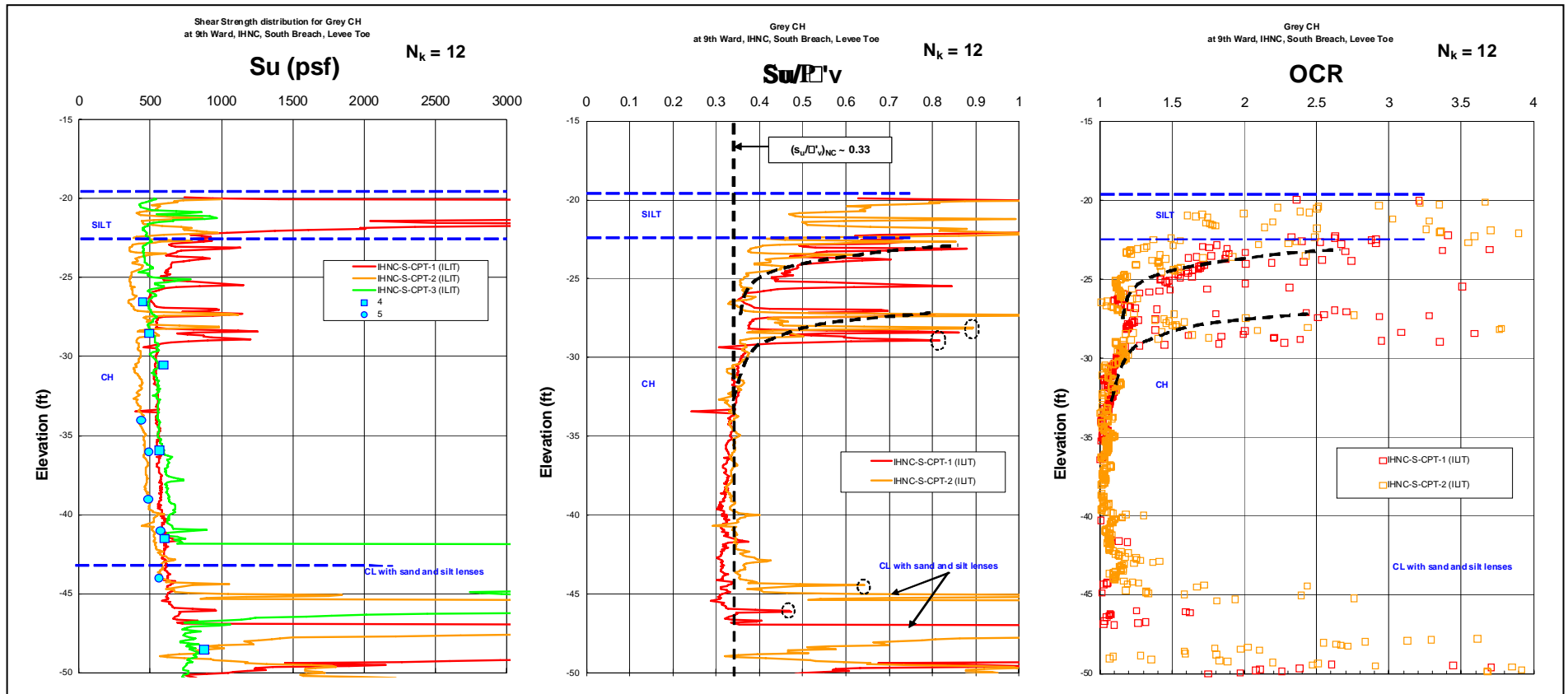


Figure 6.25: Plots of (a) OCR vs. Depth and (b) S_u vs. Depth for the soft gray marsh clay (CH) at the inboard toe and further to the landside (not under levee embankment overburden pressure) – Lower Ninth Ward, IHNC East Bank, South Breach site.

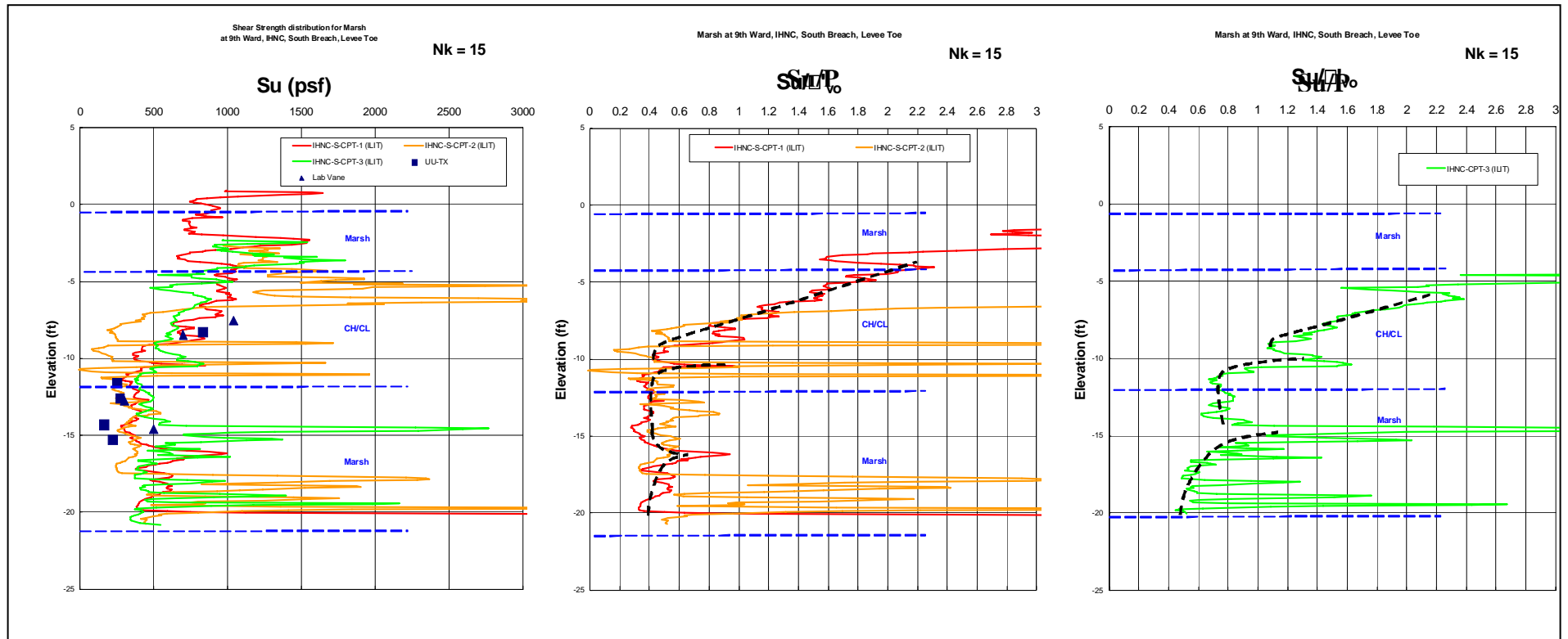


Figure 6.26: Plots of (a) OCR vs. Depth and (b) S_u vs. Depth for the shallow marsh and clay deposits at the inboard toe and further to the landside (not under levee embankment overburden pressure) – Lower Ninth Ward, IHNC East Bank, South Breach site.

ID	Soil Model	Name	Type	g_unsat [lb/ft ³]	g_sat [lb/ft ³]	k_x [ft/day]	k_y [ft/day]	nu [-]	E_ref [lb/ft ²]	c_ref [lb/ft ²]	phi [°]
CF	Mohr Coulomb	ML-silt (Compacted Fill)	Undrained	105	115	0.0028	0.00028	0.35	234000	900	0.001
Bottom		Gray Clay (bottom)	Undrained	90	90	0.0028	0.0028	0.3	155000	800	0.001

ID	Soil Model	Name	Type	g_unsat [lb/ft ³]	g_sat [lb/ft ³]	k_x [ft/day]	k_y [ft/day]	lambda*	kappa*	n_ur [-]	KDnc [-]	M [-]	c_ref [lb/ft ²]	phi @ pref [°]	OCR
M2	Soft Soil	Marsh 2	Undrained	80	80	28.3	2.8	0.21	0.033	0.15	0.60	1.90	0	36	1.1
M1		upper marsh	Undrained	80	80	28.3	2.8	0.2105	0.033	0.15	0.60	1.90	0.0001	36	2.25
ML		silt	Undrained	85	85	28.32	2.8	0.1	0.02	0.15	0.61	1.27	0.001	23	2
CH2		Gray Clay (CH)	Undrained	90	95	0.00028	0.00028	0.1684	0.03	0.15	0.63	1.24	0.0001	22	2.25-1.70
CH3		Gray Clay (CH)	Undrained	90	95	0.00028	0.00028	0.1684	0.03	0.15	0.63	1.24	0.001	22	1.1

ID	Name	Type	EA [lb/ft]	EI [lb/ft ²]	w [lb/ft ²]	nu [-]	M_p [lb/ft]	N_p [lb/ft]
1	I-wall	Elastic	6.32E+08	54290000	150	0.22	1.00E+15	1.00E+15
2	Sheetpile (Z-22)	Elastic	1.87E+08	16400000	22	0.22	1.00E+15	1.00E+15

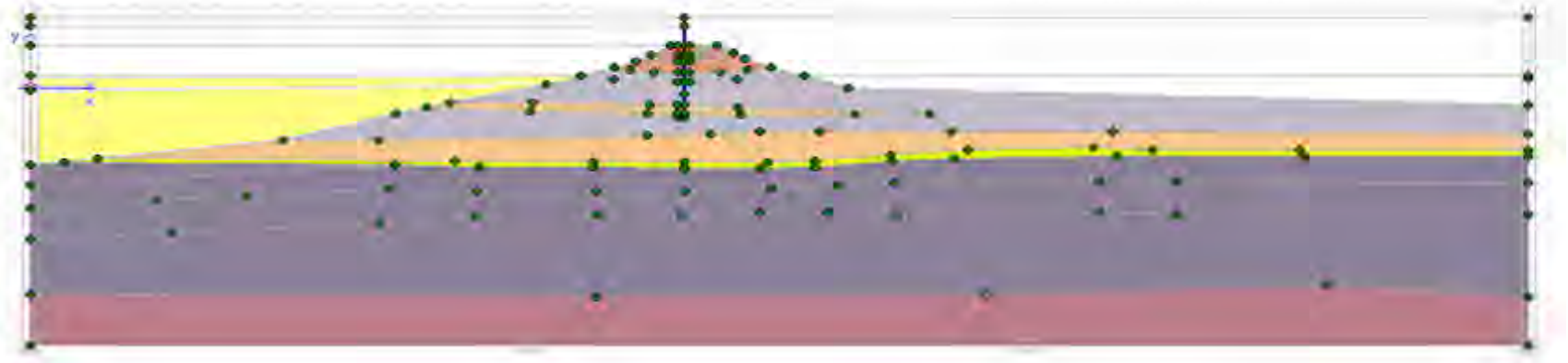


Figure 6.27: Geometry and input parameters for FEM (PLAXIS) stability analyses for Lower Ninth Ward, IHNC East Bank, South Breach.

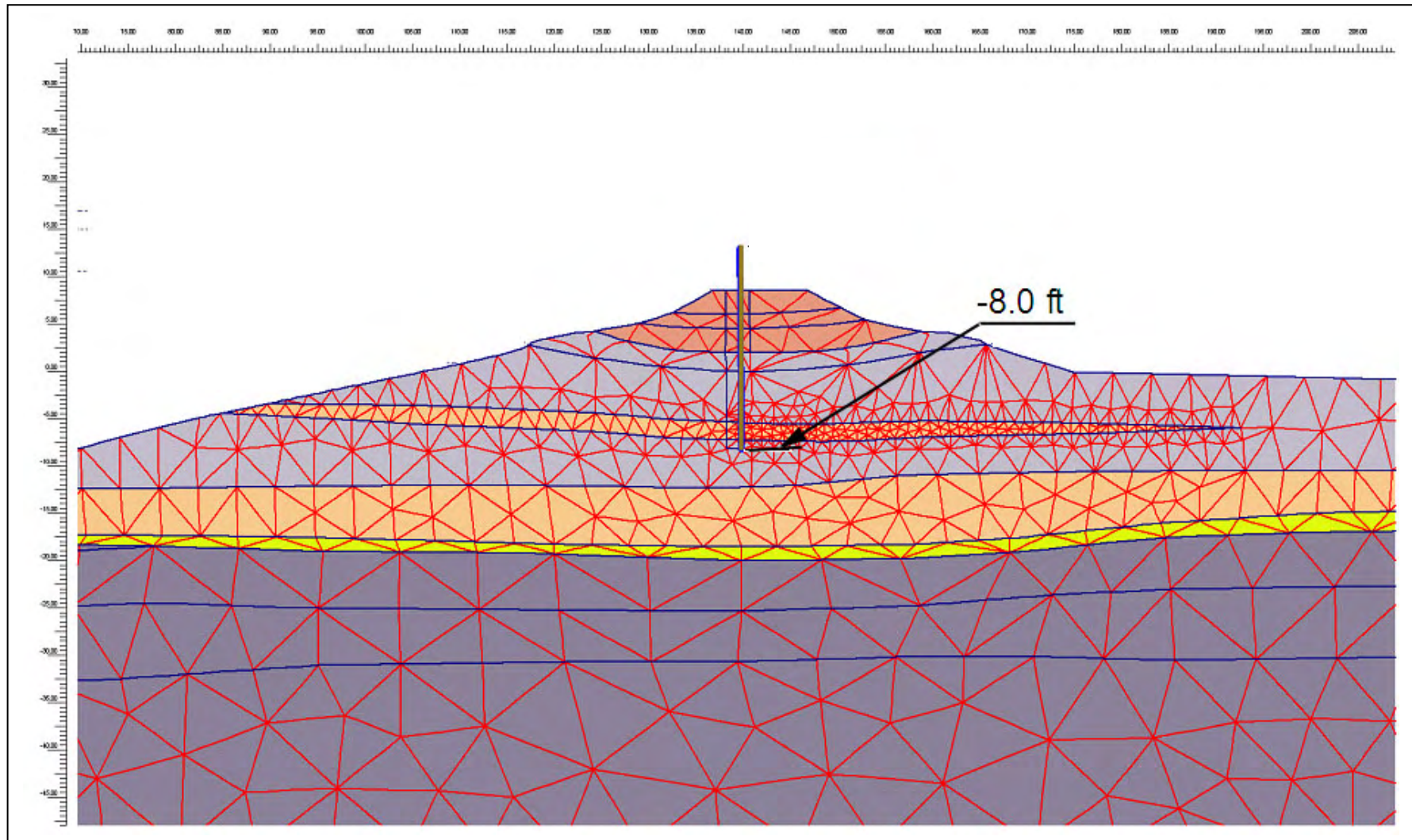


Figure 6.28: Deformed mesh after modeling staged construction for the levee and allowing for consolidation for Lower Ninth Ward, IHNC East Bank, South Breach.

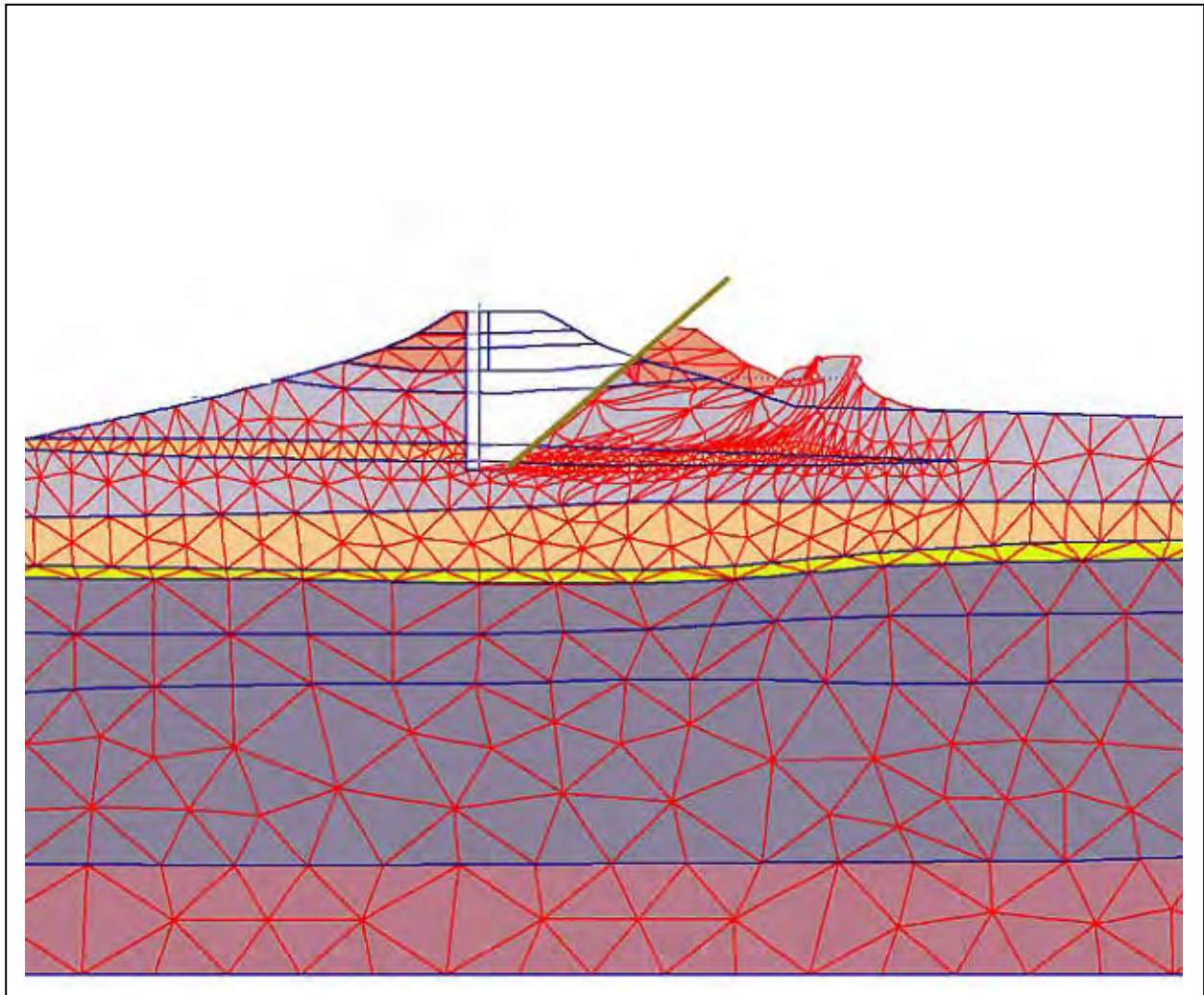


Figure 6.29: Deformed mesh for storm surge elevation of +13.5ft (MSL) for Lower Ninth Ward, East Bank, South Breach.

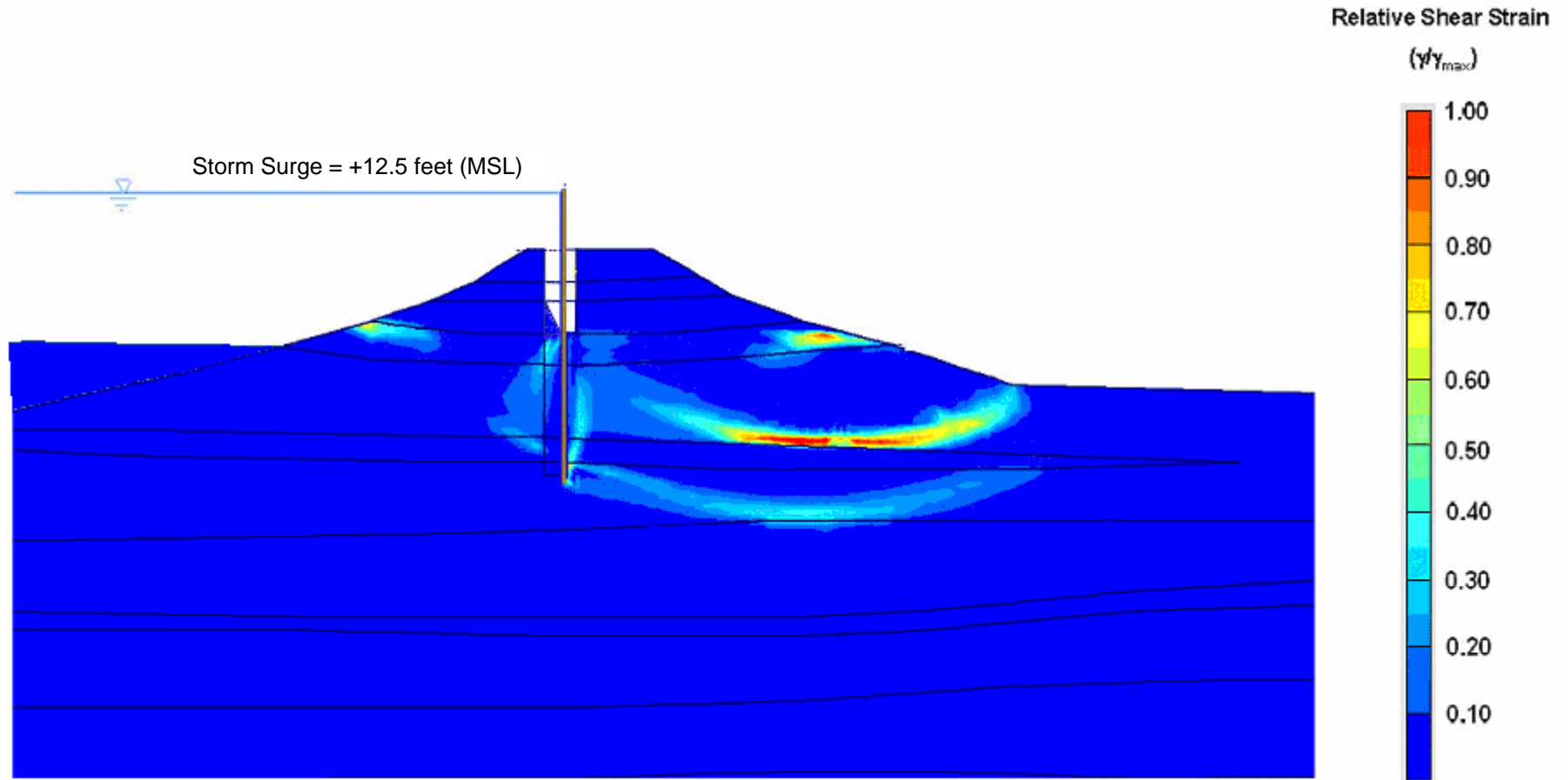


Figure 6.30: Normalized shear strain contours (shear strain divided by strain to failure) for a storm surge at Elev. + 12.5 feet (MSL) at the Lower Ninth Ward, IHNC East Bank, South breach site; gapping at outboard toe of floodwall is developed fully through the embankment fill material.

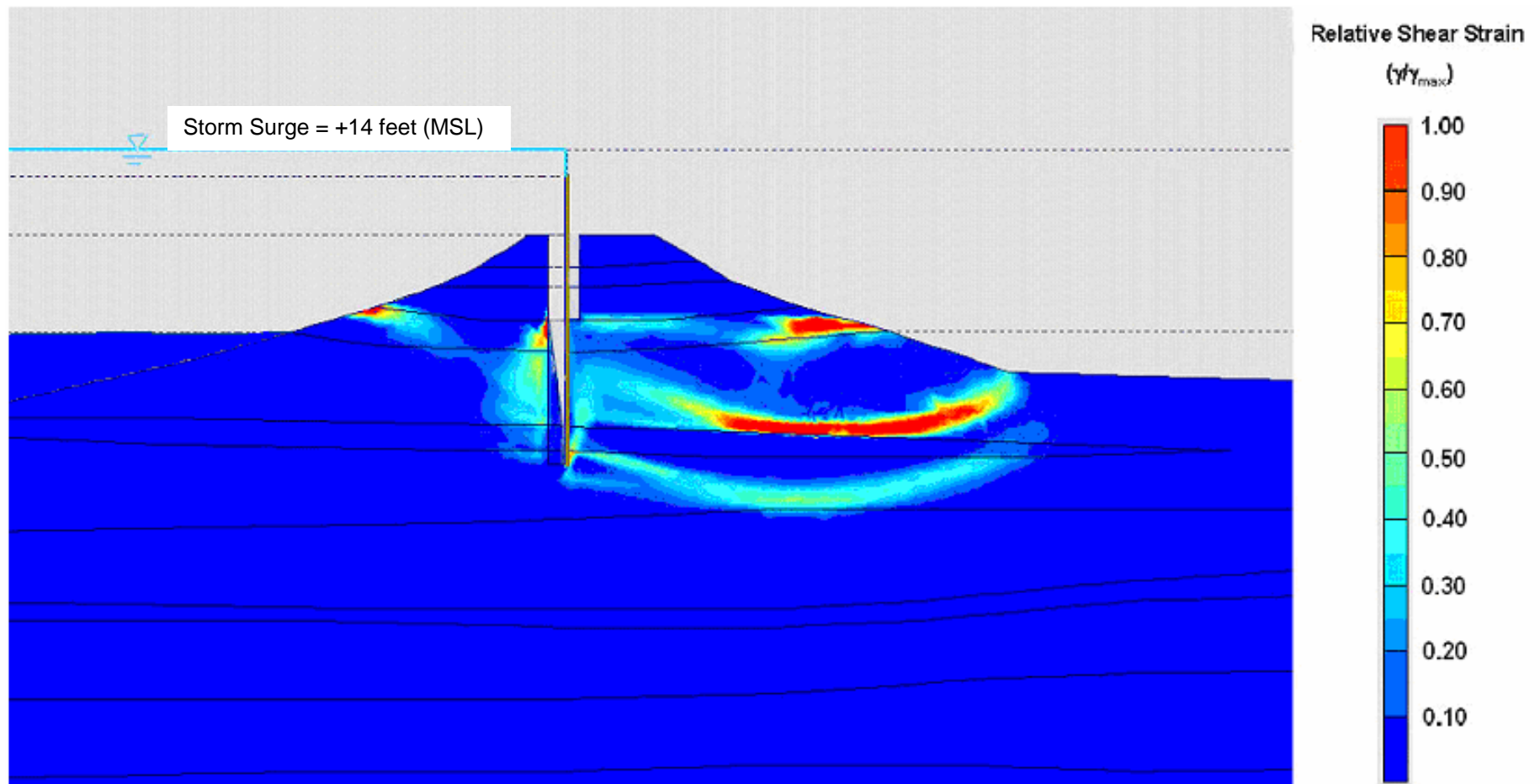


Figure 6.31: Normalized shear strain contours (shear strain divided by strain to failure) for a storm surge at Elev. + 14 feet (MSL) at the Lower Ninth Ward, IHNC East Bank, South breach site; gapping at outboard toe of floodwall is fully developed.

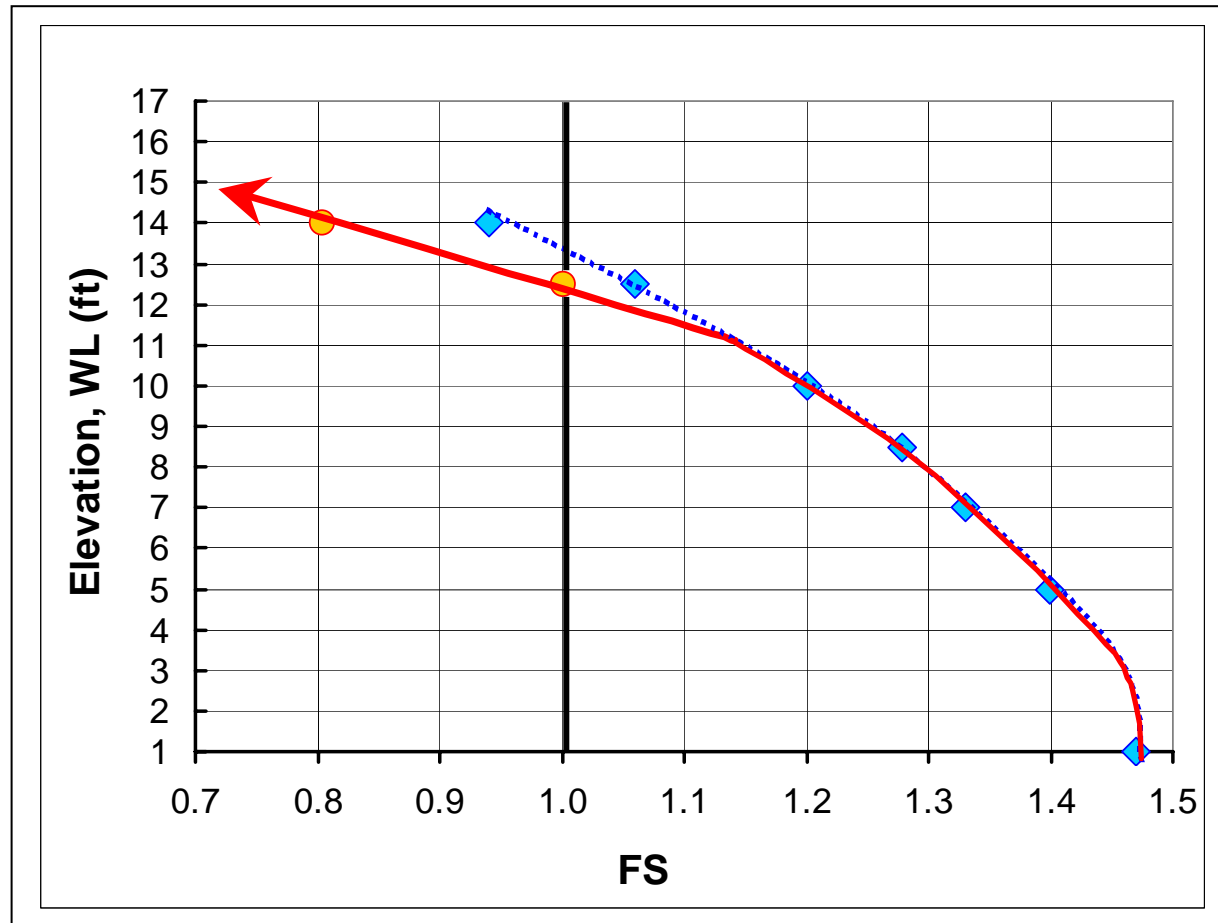


Figure 6.32: Calculated Factors of Safety for two modes based on PLAXIS analyses of the Lower Ninth Ward South breach site (east bank IHNC) for various canal water elevations; showing the best-estimated path to failure.

MATERIAL	PARAMETER						
	γ (pcf)	ϕ	c (psf)	Kh (ft/hr)	Kh (cm/s)	Kv/Kh	Θ *
Fill	105	0	900	1.17E-04	9.91E-07	1	0.35
Upper CH	95	0	800	2.00E-04	1.69E-06	0.333	0.35
Upper Marsh	85	28	0	1.10E+00	9.31E-03	0.25	0.5
OC Grey CH	95	0	500	2.00E-04	1.69E-06	0.333	0.35
NC Grey CH	95	0	Su/p: 0.28	2.00E-04	1.69E-06	0.333	0.35
Lower Marsh	85	28	0	1.10E+00	9.31E-03	0.25	0.5
Silt	110	0	600	1.17E-04	9.91E-07	0.333	0.41
Lean Clay	100	0	600	2.00E-04	1.69E-06	0.333	0.38
Sands	120	30	0	1.00E+00	8.5E-03	0.5	0.42
Gaps				100		10	1

* Fredlund et al., Green and Corey, Van Genuchten

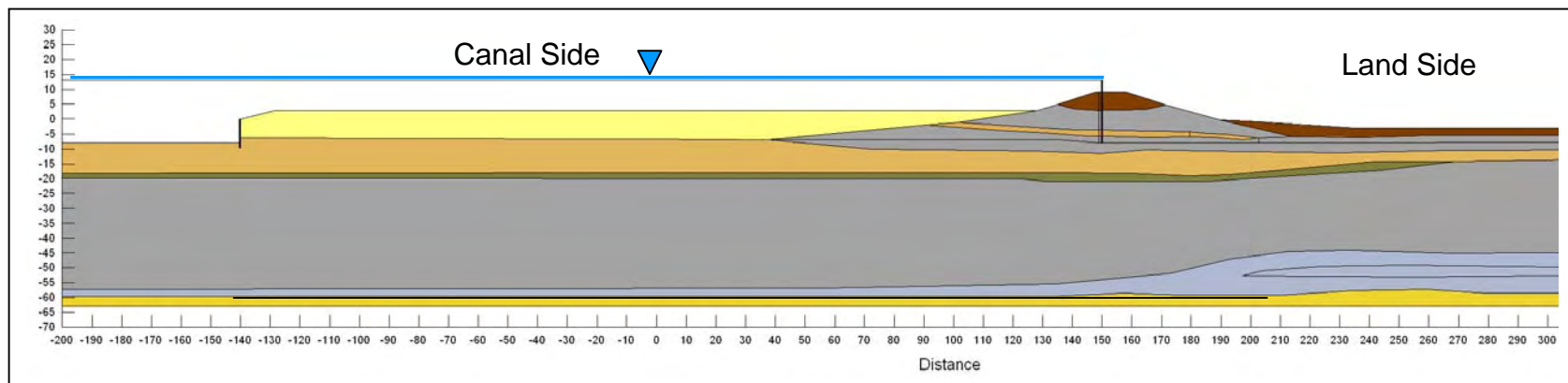


Figure 6.33: Geotechnical cross-section for analysis of the IHNC east bank, south breach.

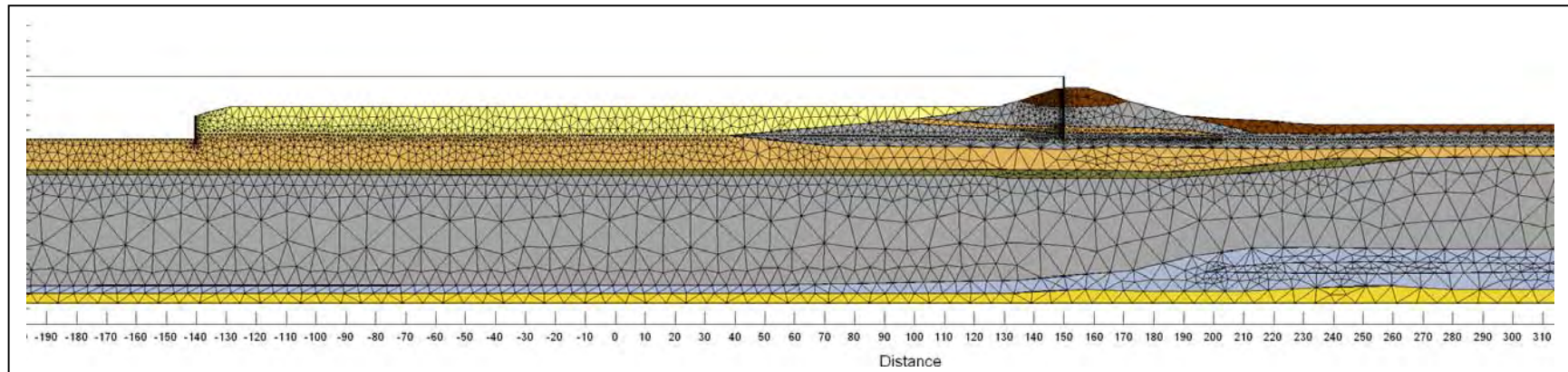


Figure 6.34: Finite difference mesh for seepage analyses for IHNC east bank, south breach.

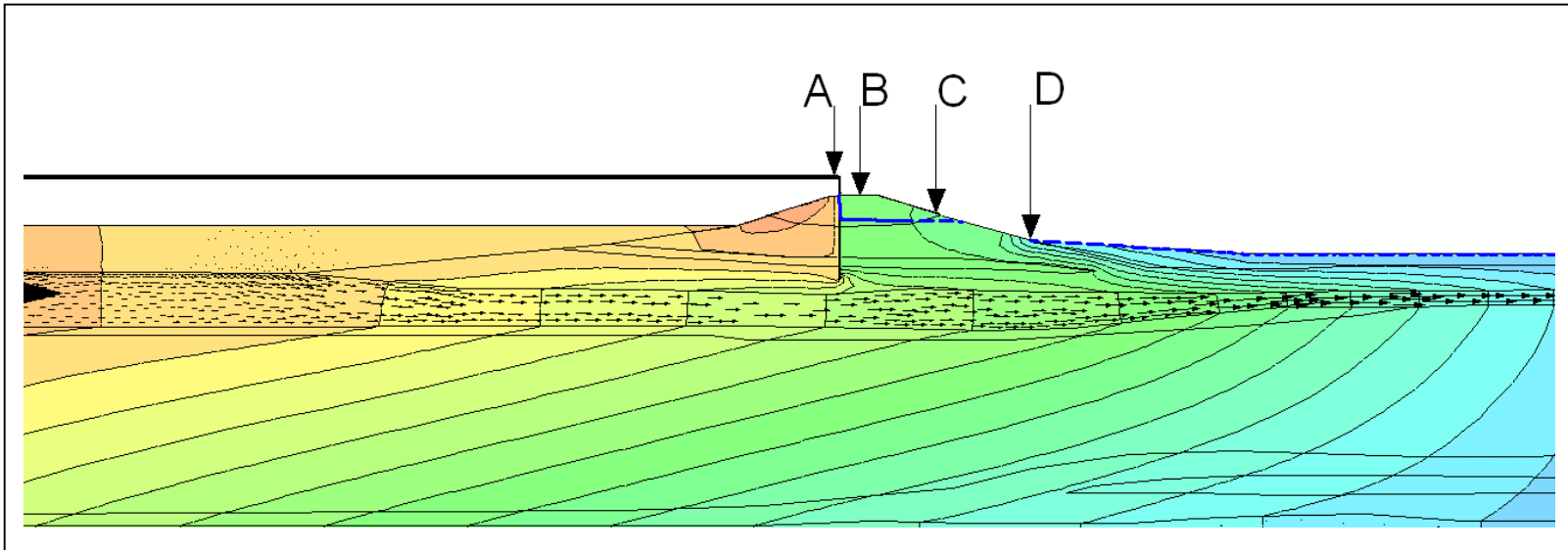


Figure 6.35: Flow net for the south breach on IHNC; storm surge at 14.4ft (MSL). Head contours at 1-foot intervals of head.

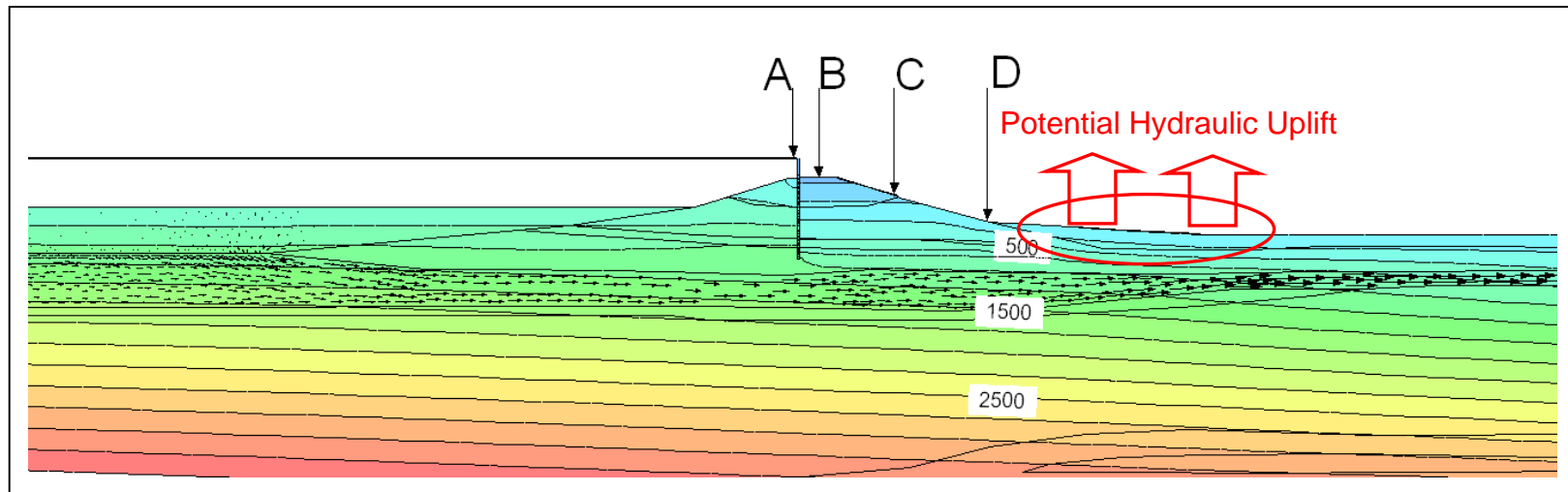


Figure 6.36: Pressure contours for the south breach on IHNC. Storm surge at 14.4ft (MSL).

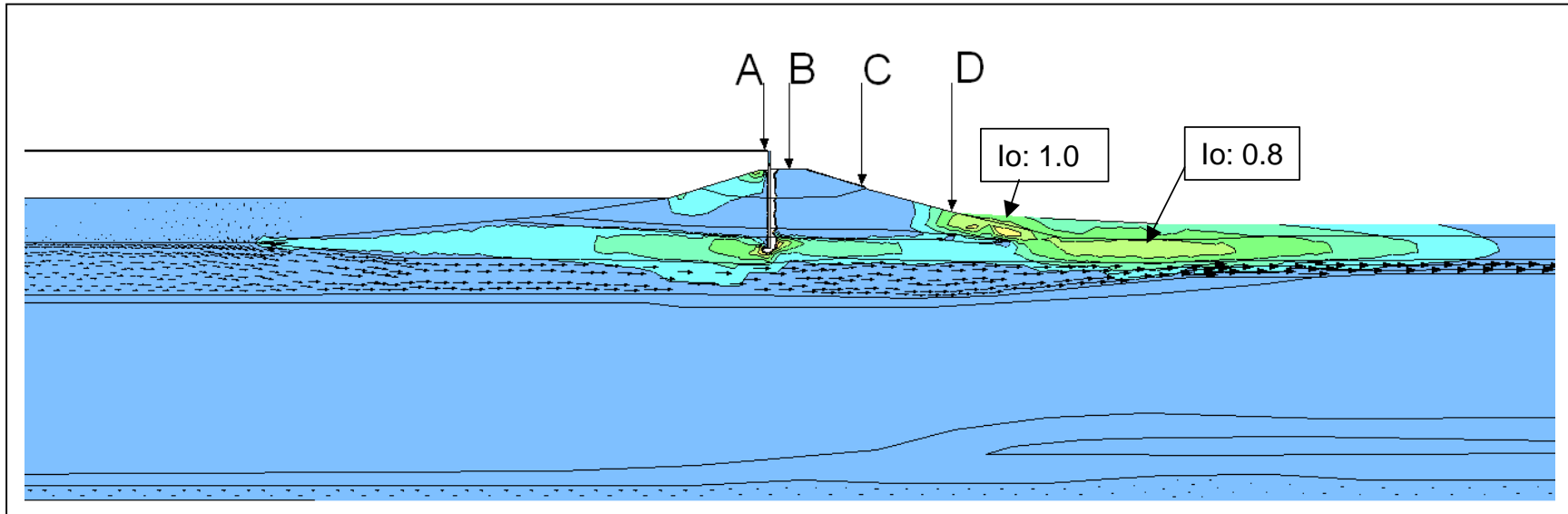


Figure 6.3 7: Hydraulic gradients for the south breach on IHNC east bank; storm surge at 14.4ft (MSL). Maximum exit gradient at the levee toe is $i_o \approx 0.8$ to 1.0.

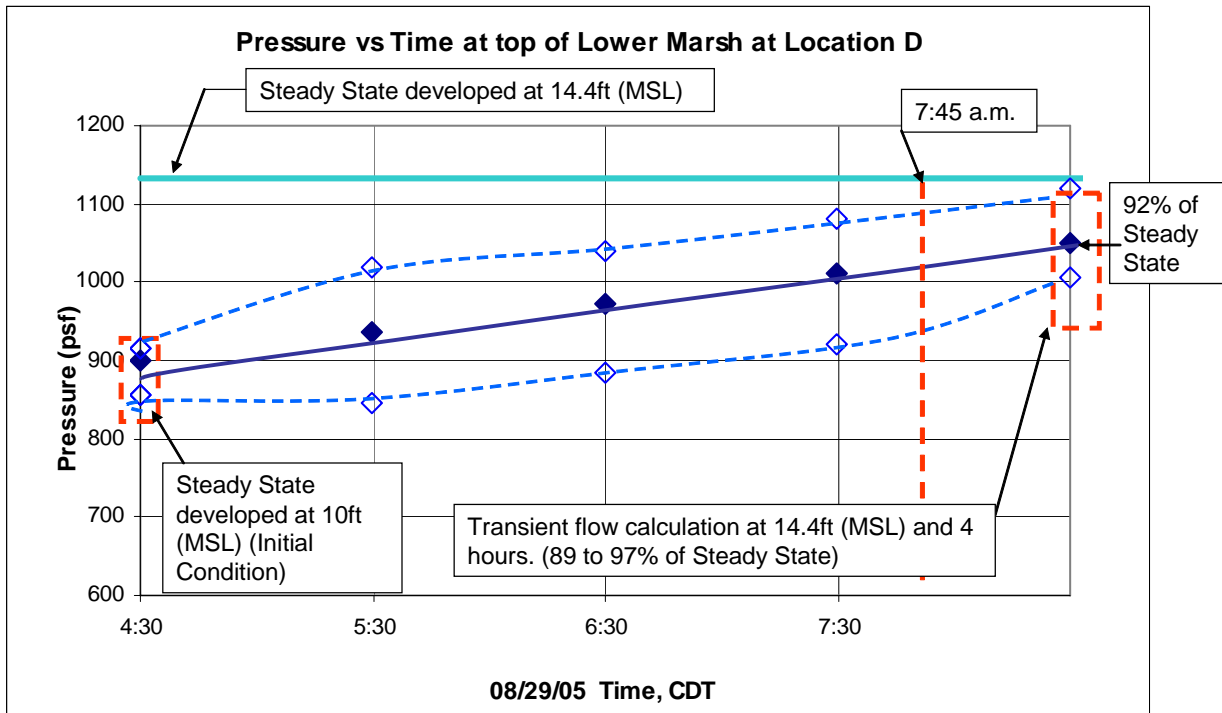


Figure 6.38: Transient flow pore pressure generation for the south breach on IHNC east bank.

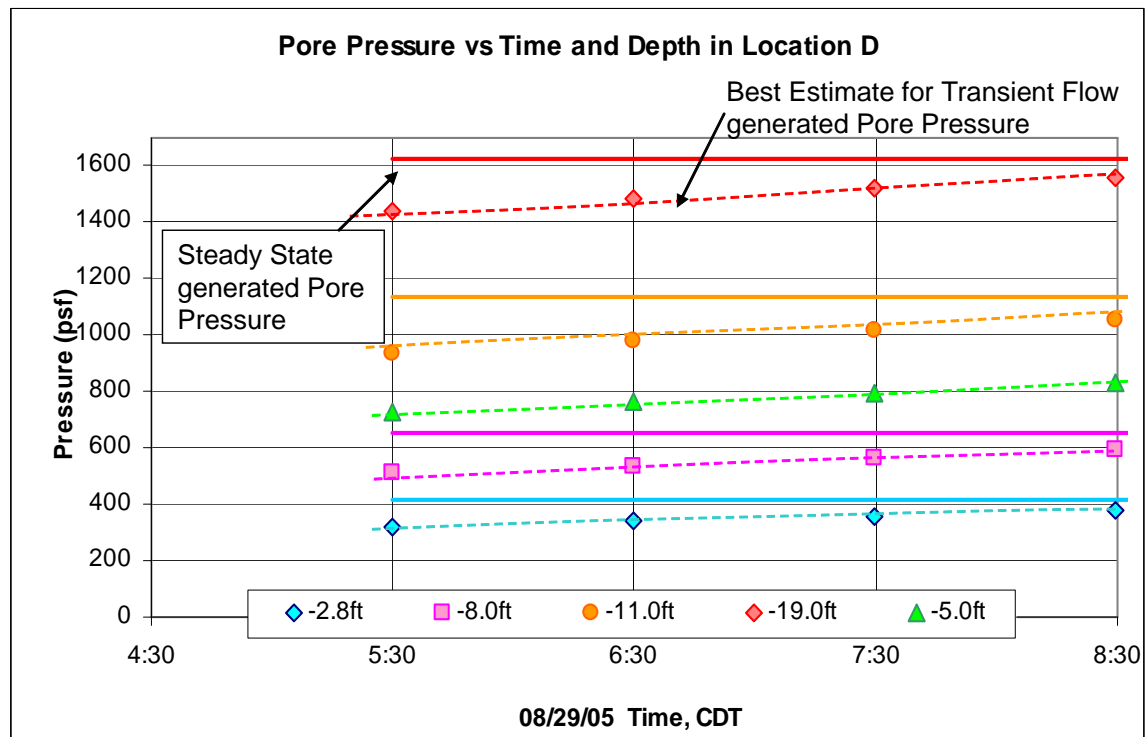
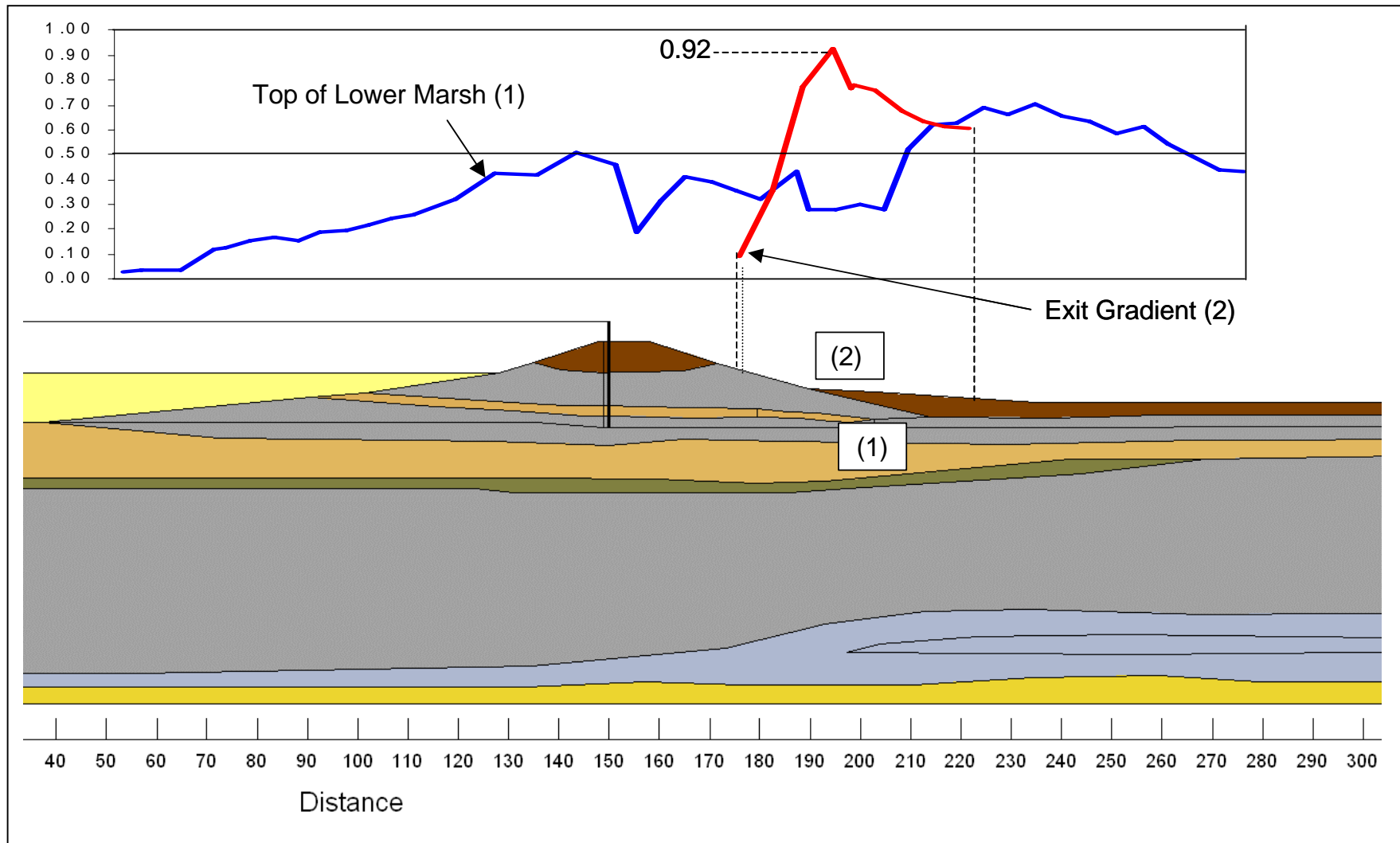


Figure 6.39: Pore pressure generation at different times and depths at the inboard toe of the south breach on IHNC east bank.



gradients at the south breach on IHNC east bank.

Figure 6.40: Hydraulic

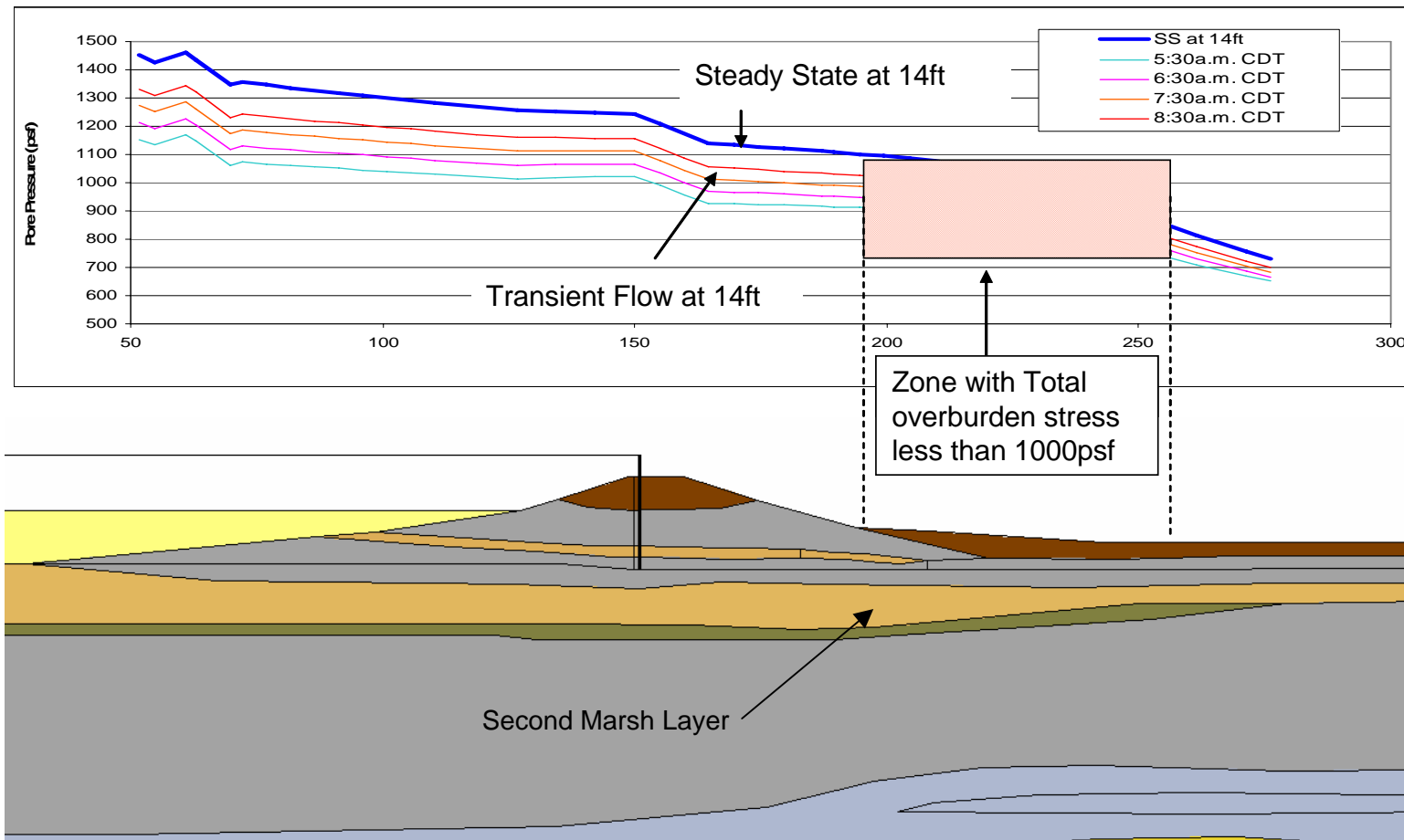


Figure 6.41: Pore pressure versus horizontal distance and time at the top of the second marsh layer; south breach on IHNC east bank.

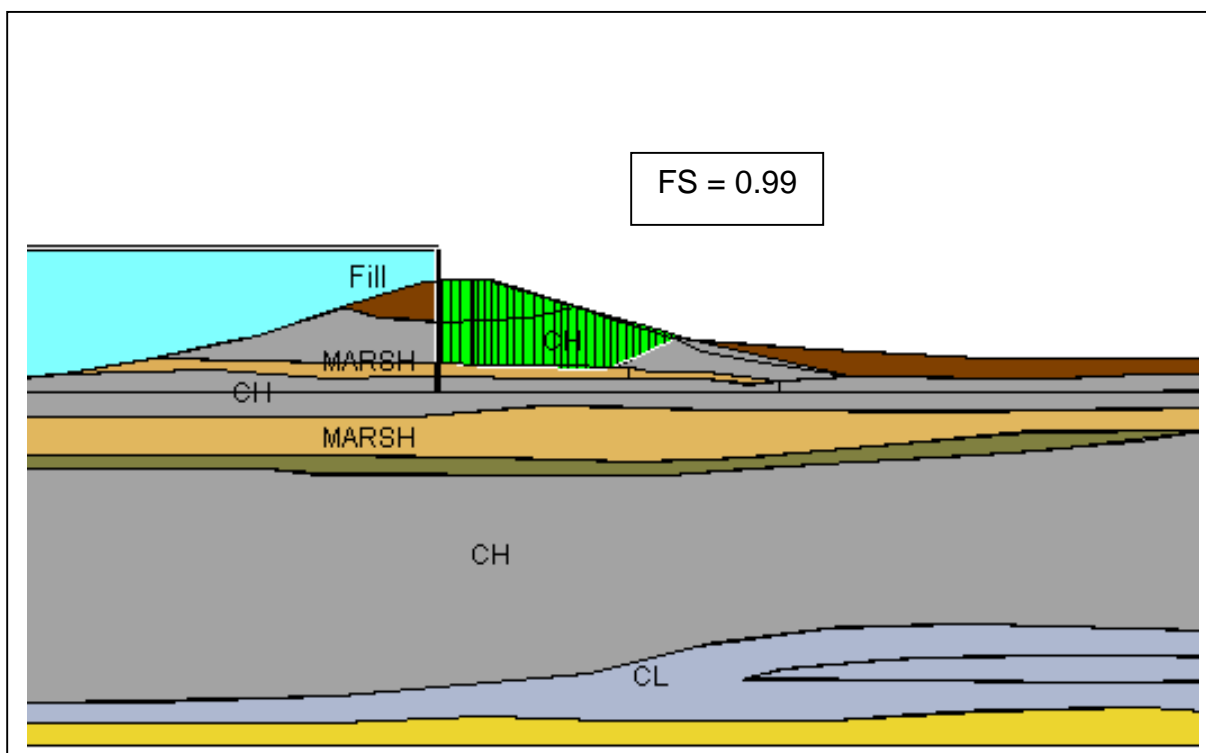


Figure 6.42: Critical failure surface for the south breach on IHNCEast bank; storm surge at 14ft (MSL), failure through upper marsh layer with gap at front of sheetpiles fully developed.

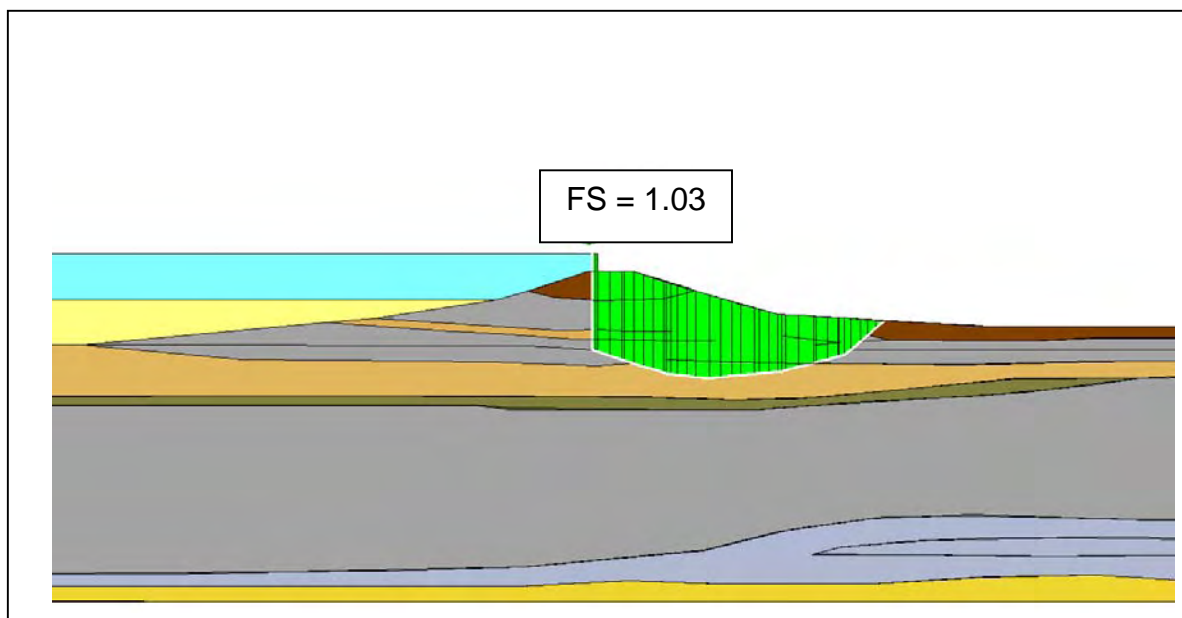


Figure 6.43: Deeper failure surface for the south breach on IHNCEast bank; storm surge at 14ft (MSL), gap at front of sheetpiles fully developed.

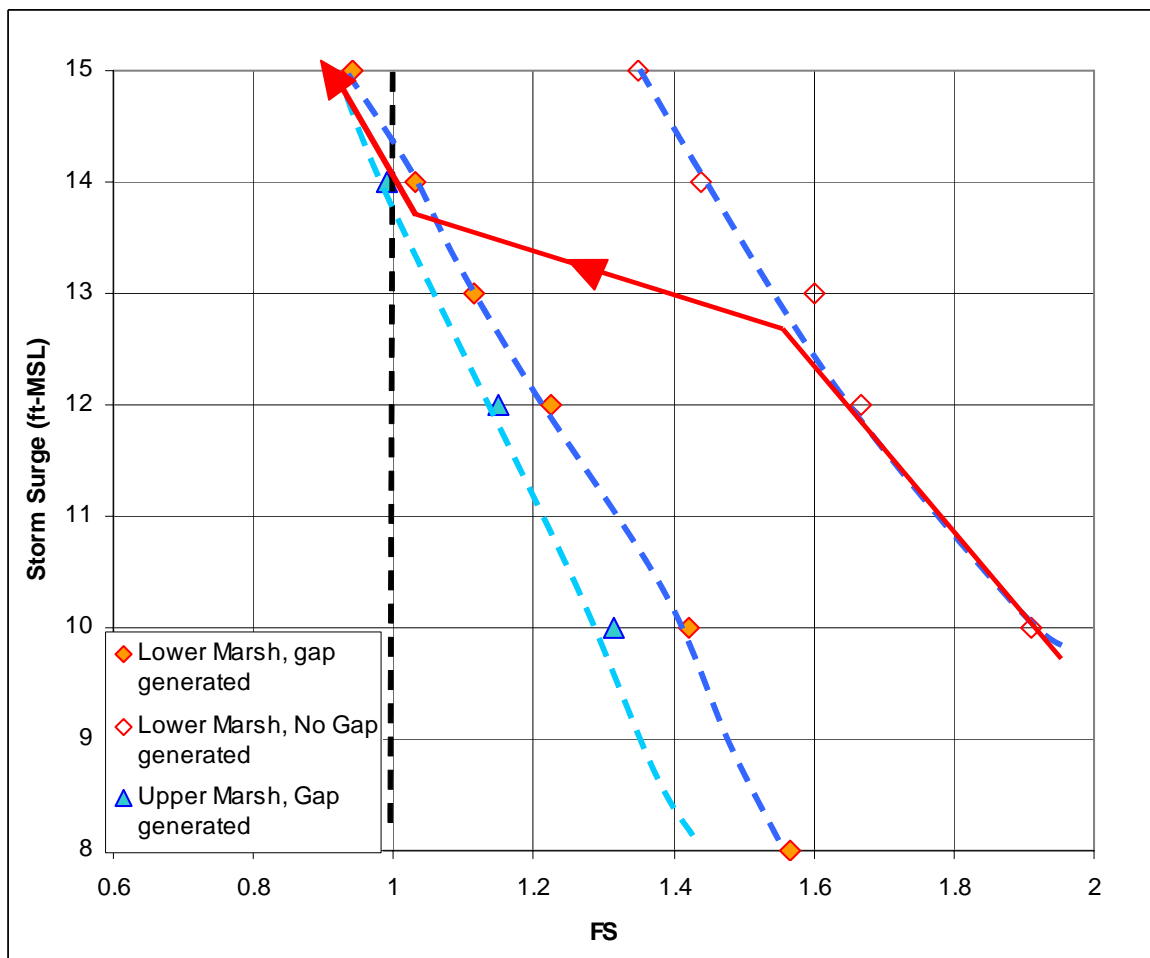


Figure 6.44: Calculated Factors of Safety for three modes based on SLOPE/W analyses of the Lower Ninth Ward South breach site (east bank IHNC) for various canal water elevations; showing the best-estimated path to failure.



Figure 6.45: Aerial view of the south breach at the east bank of the IHNC (at the west end of the Ninth Ward), showing the crevasse splay generated by reverse drainage flow.

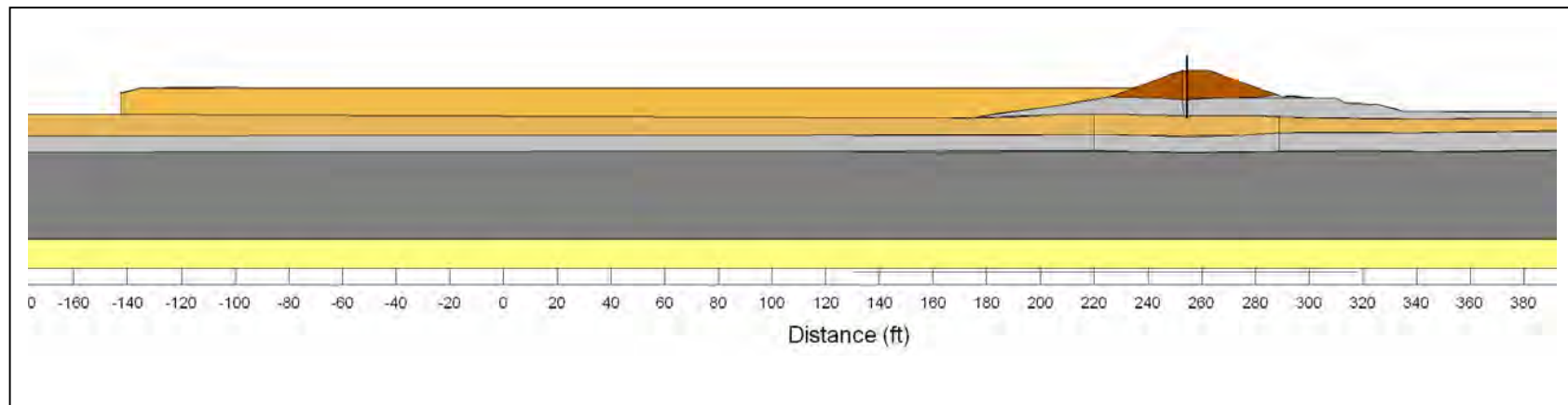
[Photograph by U.S. Army Corps of Engineers]



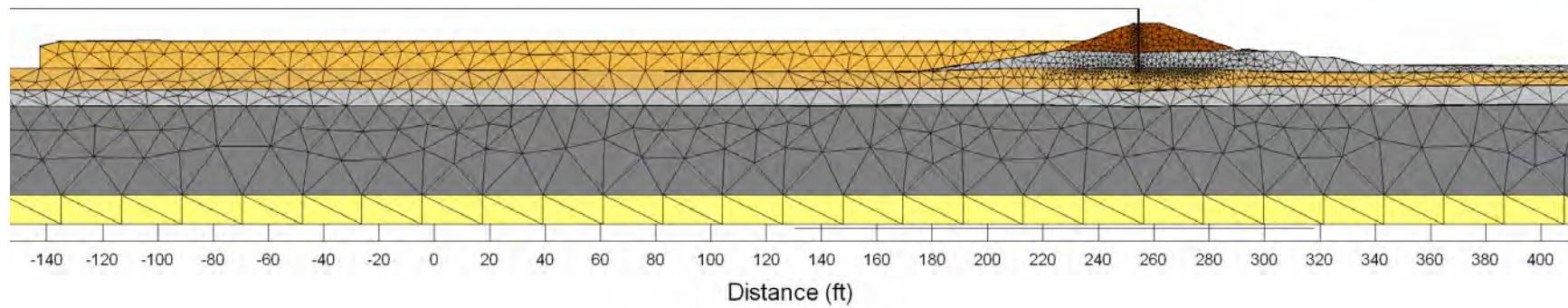
Figure 6.46: Aerial view of the partially repaired north breach on the east bank of the IHNC at the west end of the Lower Ninth Ward.

MATERIAL	PARAMETER						
	γ (pcf)	ϕ	c (psf)	Kh (ft/hr)	Kh (cm/s)	Kv/Kh	Θ *
Fill	105	0	900	1.17E-04	9.91E-07	1	0.35
CH	95	0	800	2.00E-04	1.69E-06	0.333	0.35
OC Grey CH	95	0	500	2.00E-04	1.69E-06	0.333	0.35
NC Grey CH	95	0	Su/p: 0.28	2.00E-04	1.69E-06	0.333	0.35
Marsh	85	28	0	1.10E+00	9.31E-03	0.25	0.5
Lean Clay	100	0	600	2.00E-04	1.69E-06	0.333	0.38
Sands	120	30	0	1.00E+00	8.5E-03	0.5	0.42
Gaps				100		10	1

* Fredlund et al, Green and Corey, Van Genuchten



e 6.47: Geotechnical cross-section for analysis of the IHNC east bank, north breach.



Finite difference mesh for seepage analyses for IHNC east bank, north breach.

Figure 6.48:

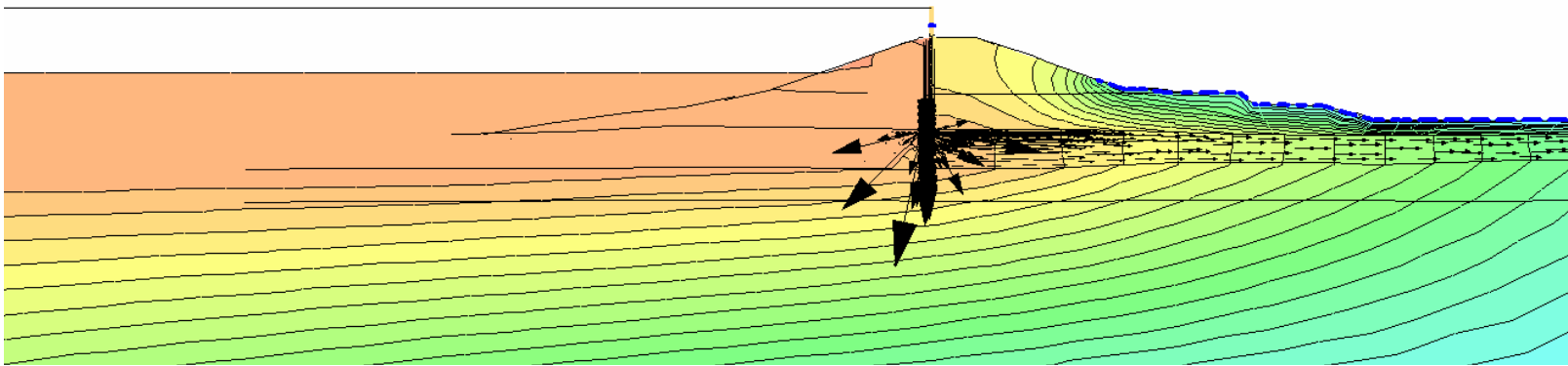


Figure 6.49: Flow net generation for the north breach on IHNC east bank. Storm surge at +14.4ft (MSL). Head contours at 1 foot intervals of hydraulic head.

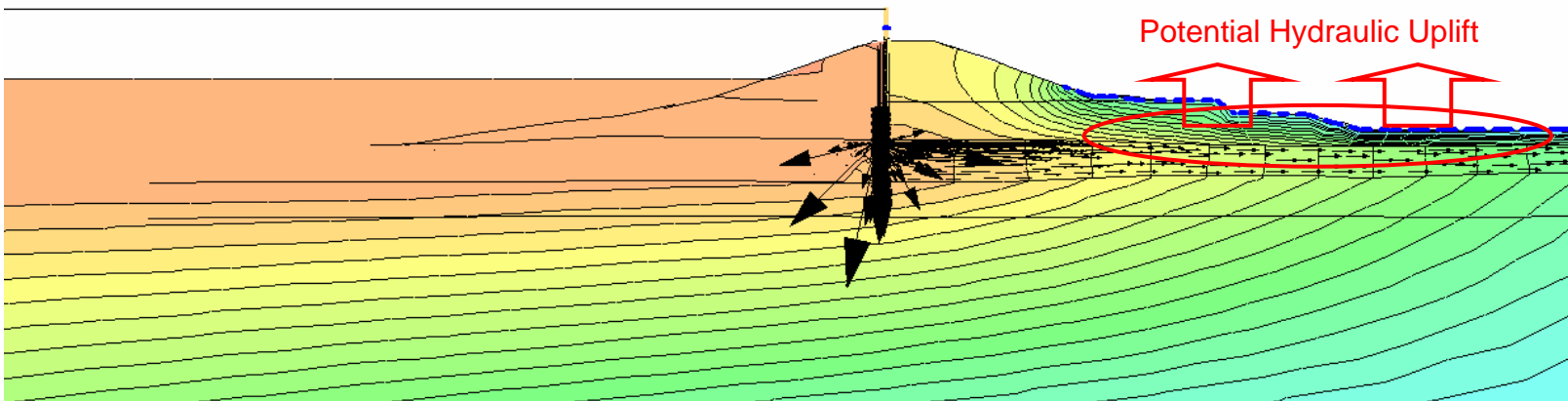


Figure 6.50: Pressure contours for the north breach on IHNC east bank. Storm surge at +14.4ft (MSL). Pore pressure contours at intervals of 62.4 lb/ft

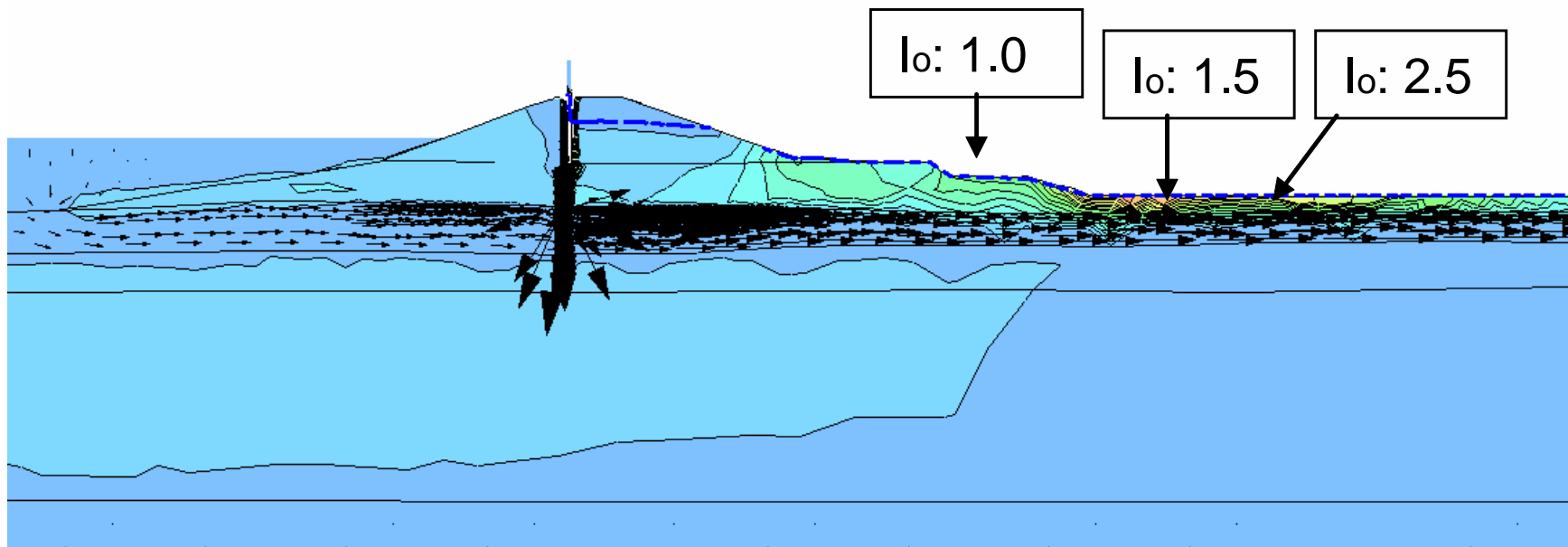


Figure 6.51: Hydraulic gradients for the north breach on IHNC. Storm surge at +14.4ft (MSL). Maximum exit gradient on the upper levee toe is $i_o \approx 1.0$, and $i_o \approx 1.5$ to 2.5 at the lower toe.

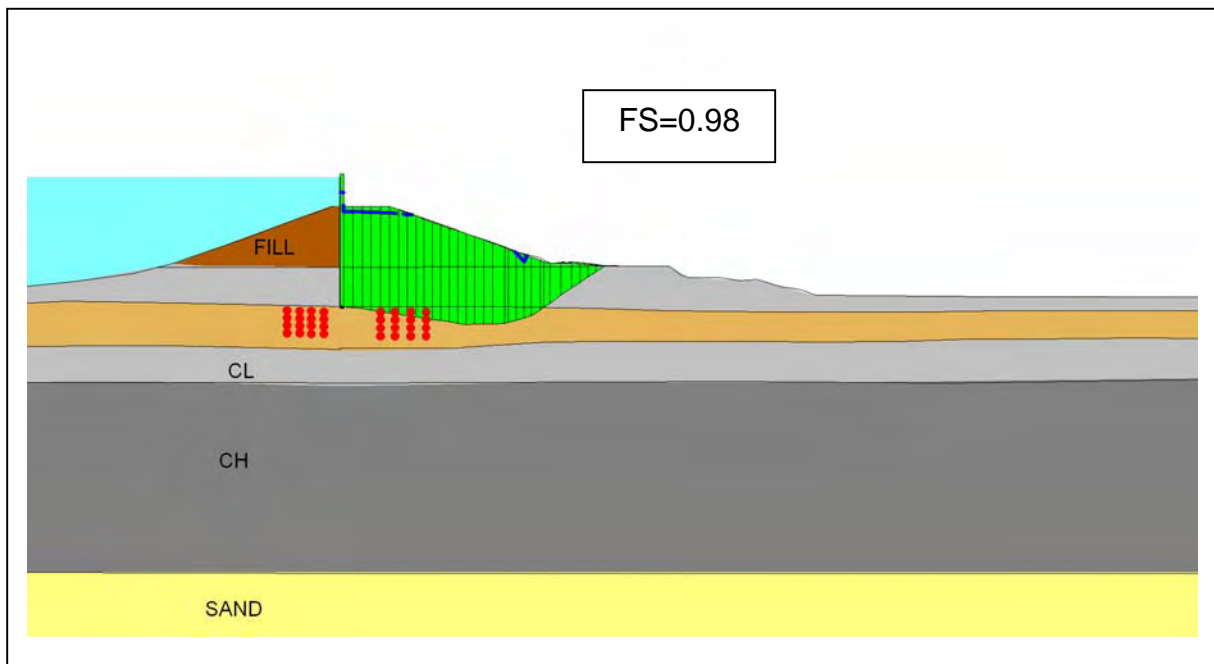


Figure 6.52: Critical potential stability failure surface for the north breach on IHNC. Storm surge at +14 ft (MSL).

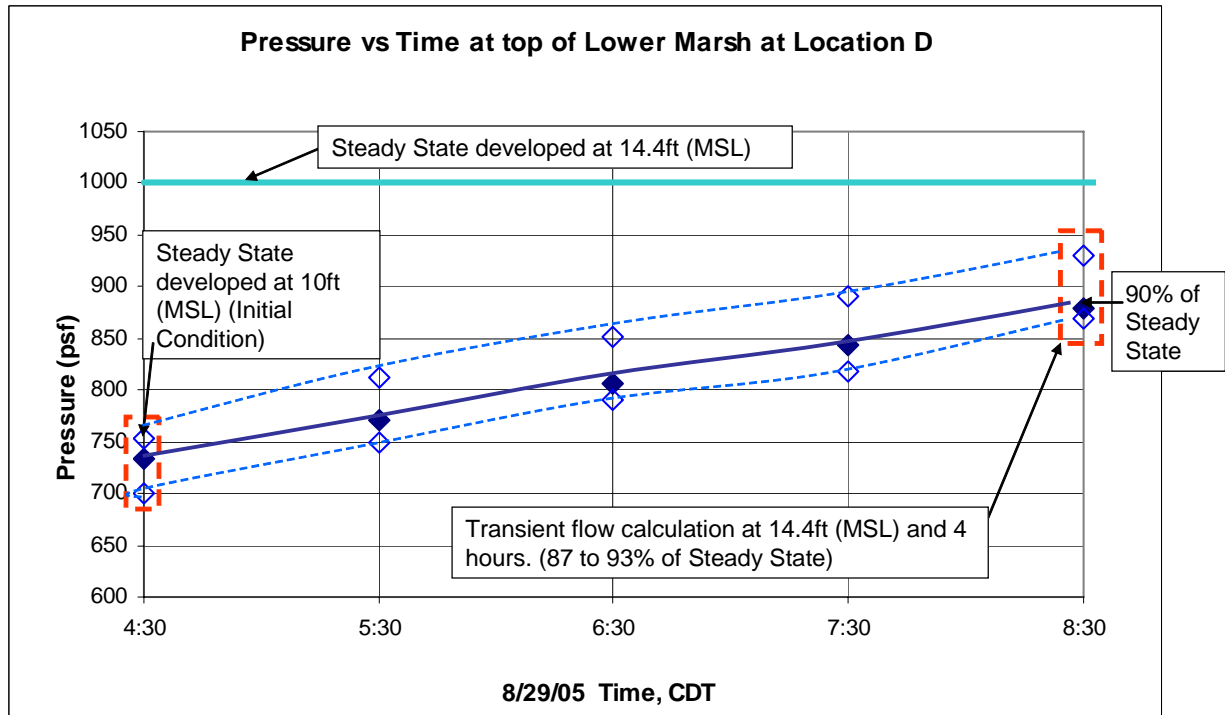


Figure 6.53: Transient flow pore pressure generation for the north breach on IHNC east bank.

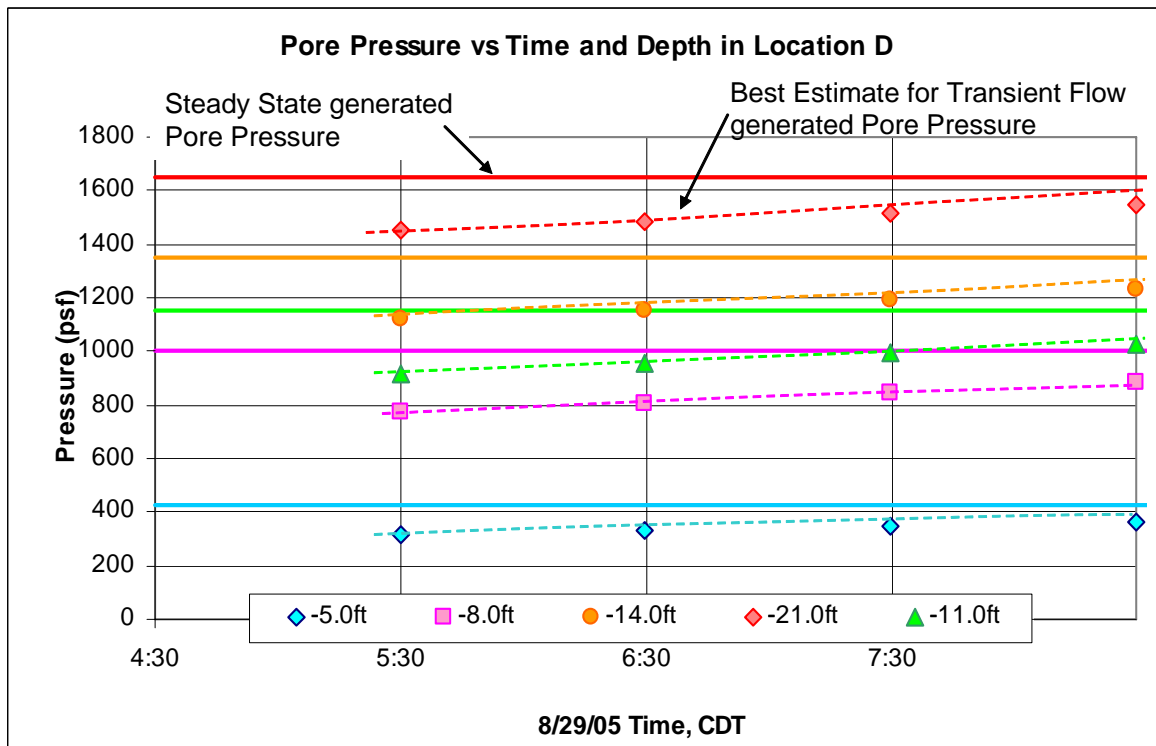


Figure 6.54: Pore pressure generation at different times and depths at the inboard toe of the north breach on IHNC east bank.

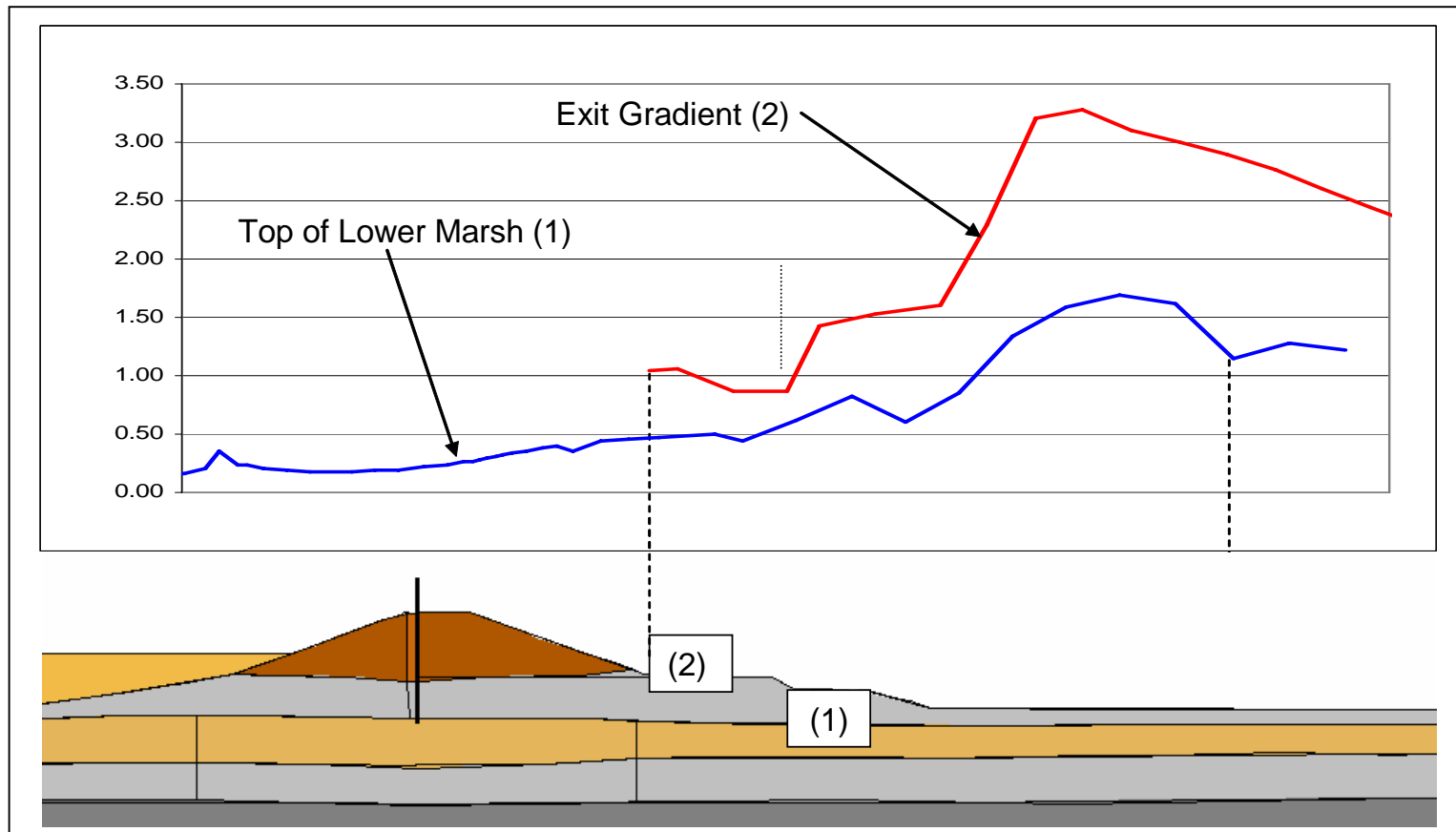


Figure 6.55: Hydraulic gradients on the south breach on IHNC east bank.

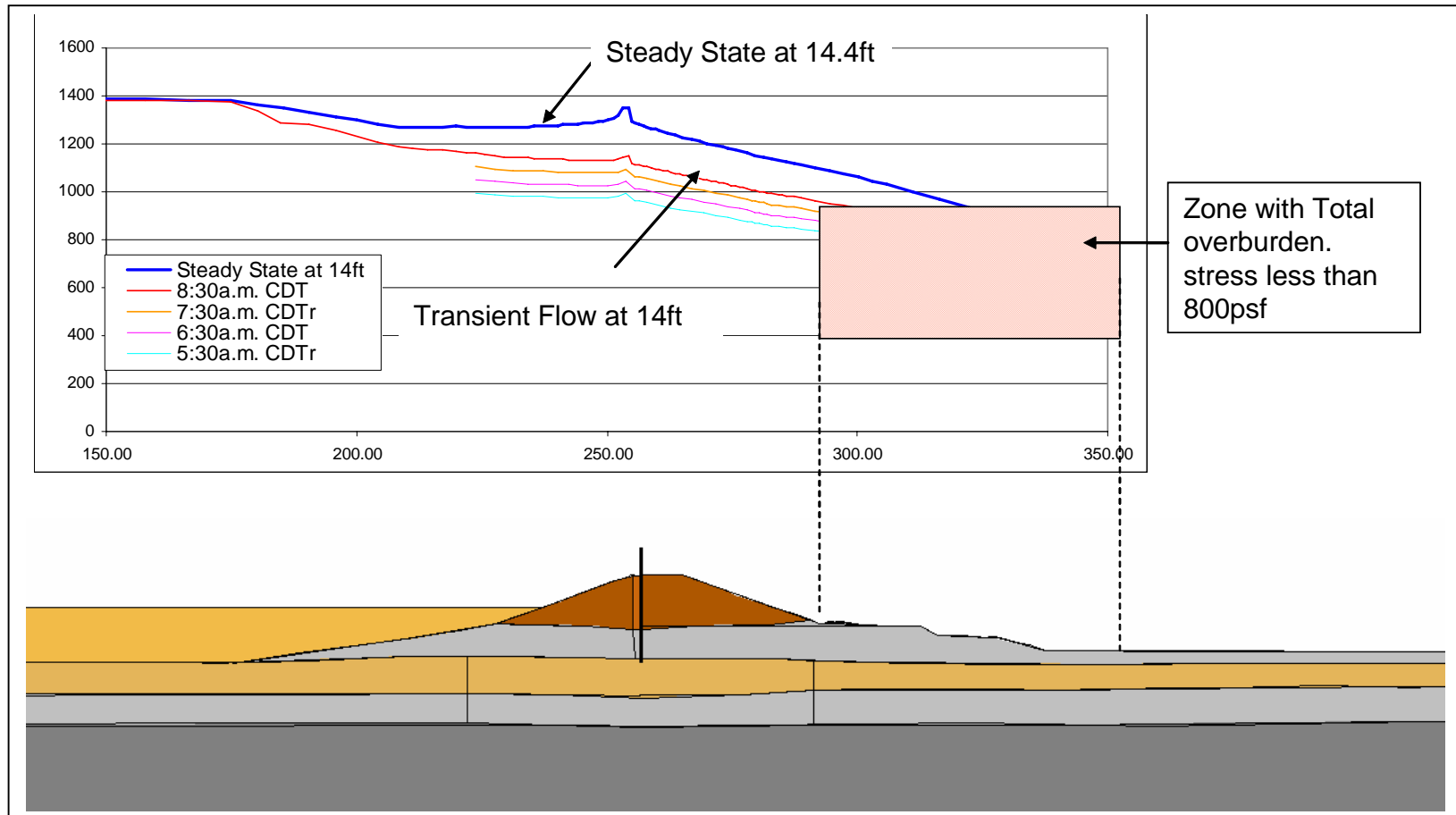


Figure 6.56: Pore pressure versus horizontal distance and time on the north breach on IHNC east bank.

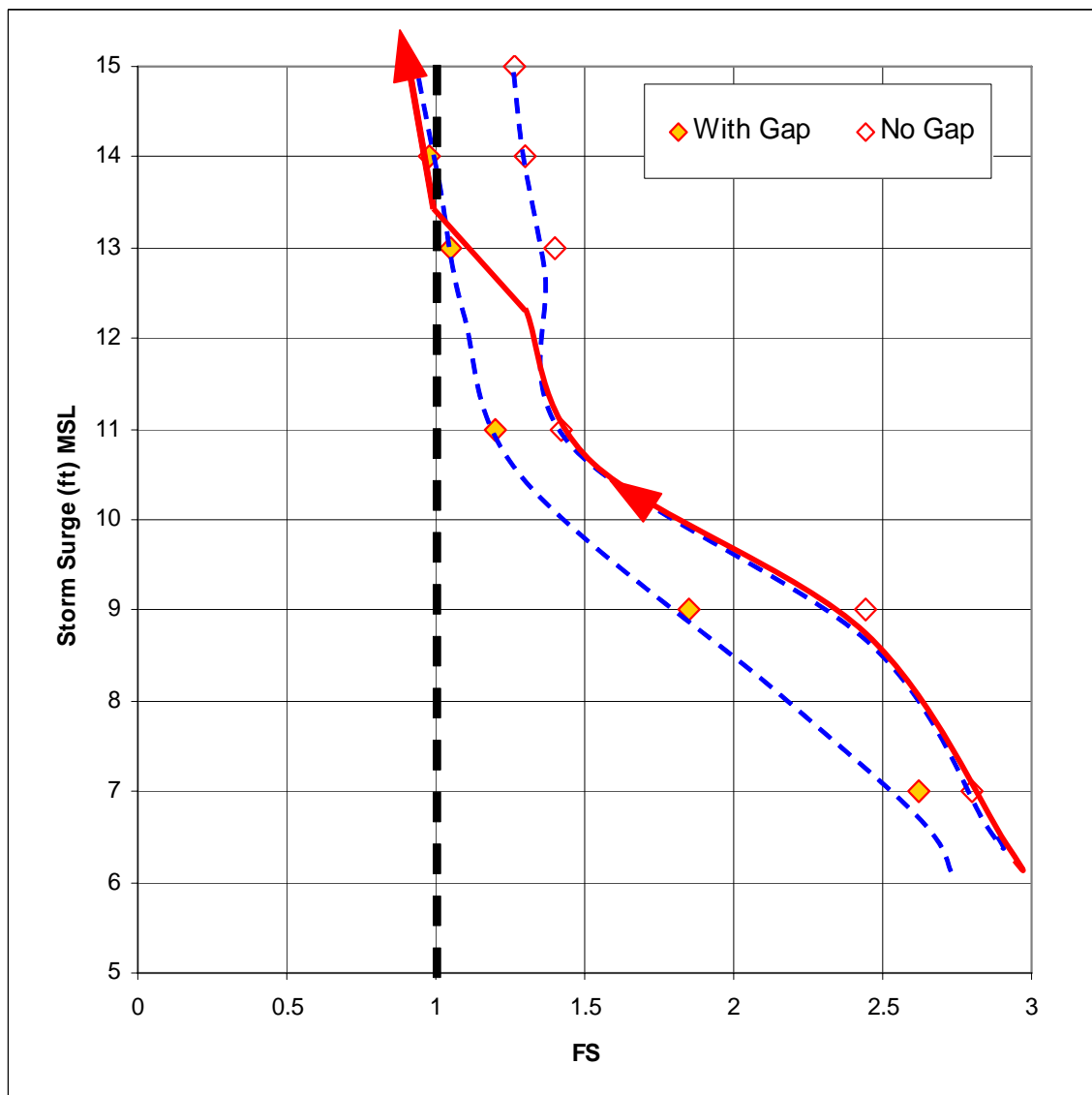


Figure 6.57: Factor of Safety vs. water elevation (ft, MSL) for the north breach, east bank of the IHNC at the west end of the Lower Ninth Ward.

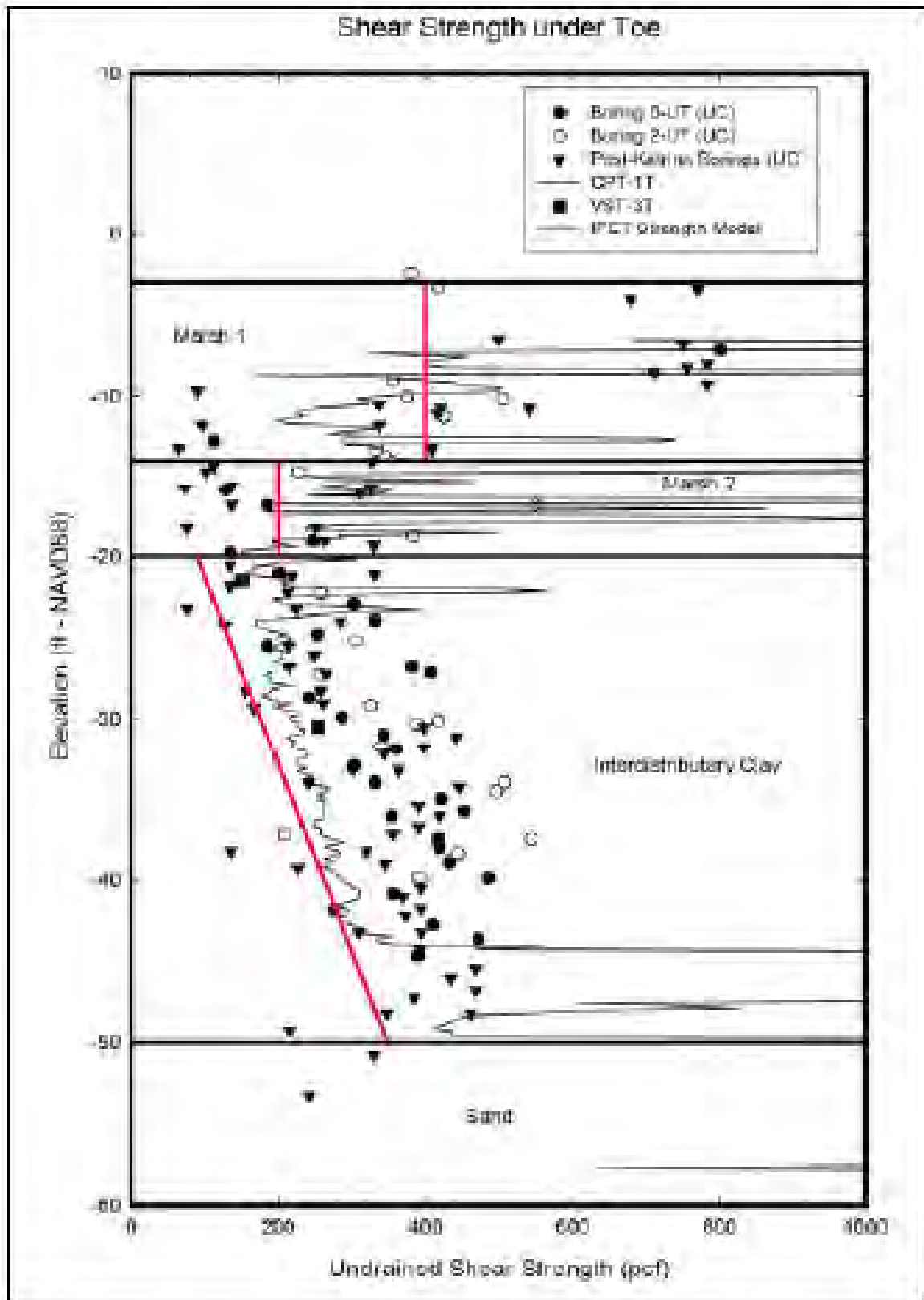


Figure 6.58: IPET shear strength profile; IHNC east bank/Lower Ninth Ward (North) breach.

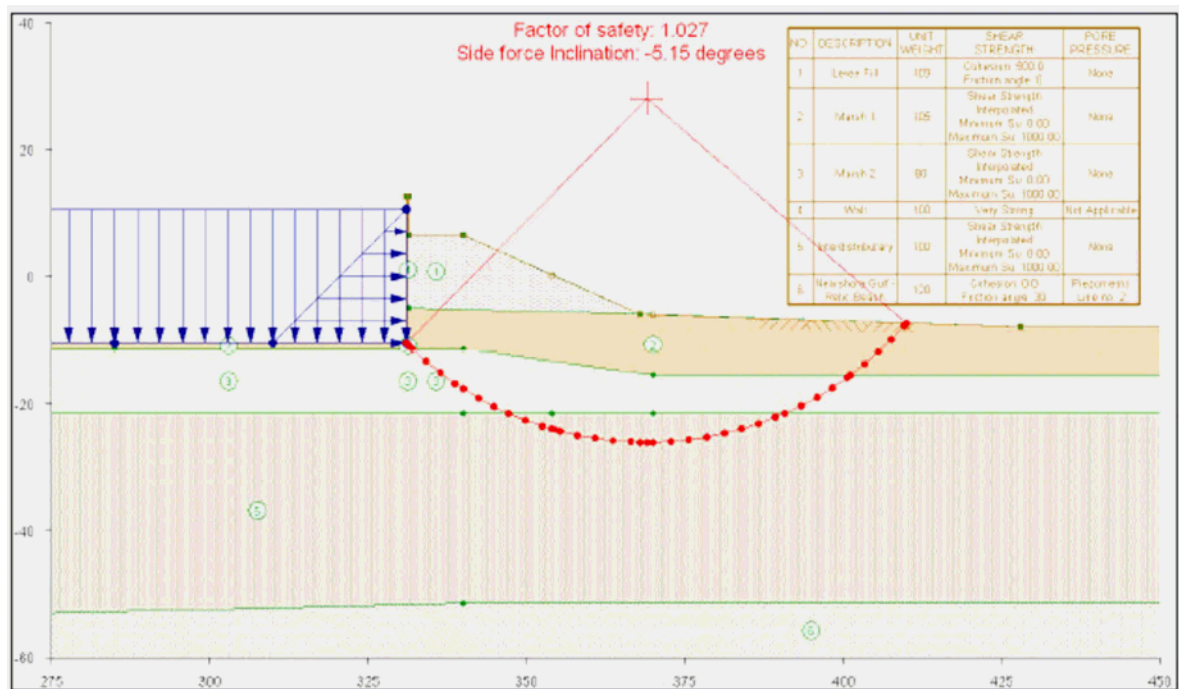


Figure 6.59: Critical limit equilibrium stability failure mode from IPET Draft Final Report;
canal water elevation at +9 feet (MSL). Factor of Safety: 1.03.

[IPET; June 1, 2006]

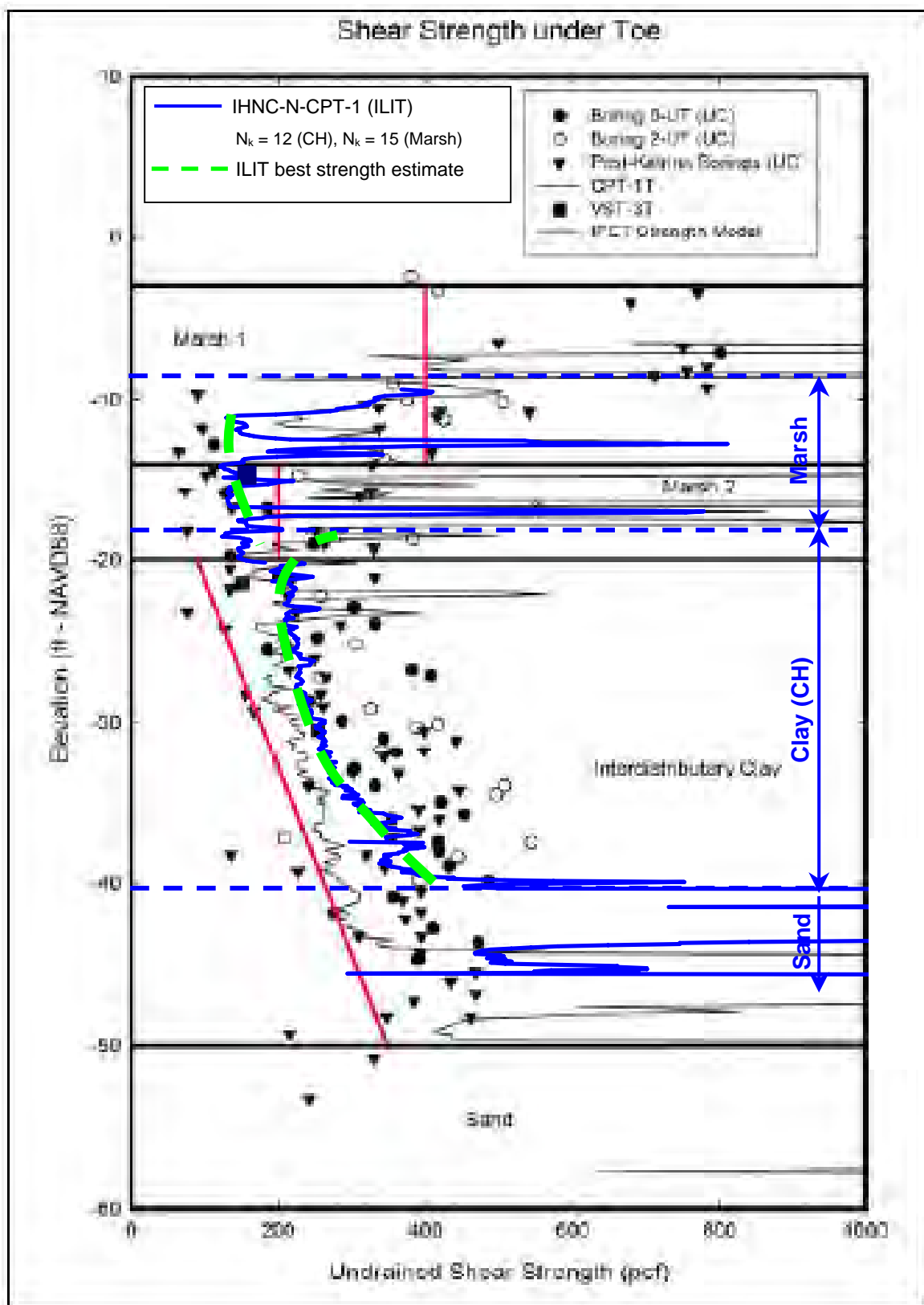


Figure 6.60: Re-interpretation of shear strength, and the ILIT shear strength profile.

CHAPTER SEVEN: THE NEW ORLEANS EAST PROTECTED AREA

7.1 Introduction

Figure 7.1 shows the New Orleans East (NEO) protected area, a contiguously ringed area that includes some of the lowest ground in the metropolitan region. This is a repeat of Figure 2.4, and the blue stars again represent levee breaches, and the red stars locations of significant levee distress. Multiple levee breaches and significant overtopping produced complete flooding of this protected area, and the resulting damage was extensive.

The New Orleans East protected area had a pre-Katrina population of approximately 96,000 people residing in over 30,000 households. Most of these residences were located in the western portion of the polder (protected area) between Lake Pontchartrain and Chef Menteur Highway (Highway I-10). The residential neighborhoods are suburban in character, with many of the homes dating to the 1960s and 1970s. Ironically, a number of these homes were built in response to the devastation inflicted by Hurricane Betsy in 1965, which had also left much of New Orleans East submerged by floodwater. This protected area also includes an industrial corridor located along its southern fringe, adjacent to the Gulf Intracoastal Waterway (GIWW) which runs adjacent to its southern edge. The eastern limits of the protected area are largely comprised of wetlands that border Lake Pontchartrain/Lake Borgne water systems and/or the swamplands between them.

The New Orleans East protected area extends over approximately 70 square miles and is bounded by Lake Pontchartrain to the north, the GIWW shipping channel to the south, and the Inner Harbor Navigation Channel (IHNC) to the west. Lake Borgne abuts the south facing levees at the southeast corner of this protected area.

Figure 7.3 shows the depths of flooding on September 2, four days after hurricane Katrina, at a time when the water levels were at equilibrium with the still slightly swollen waters of Lake Pontchartrain (Elev. ~ +1 foot, MSL), and this map of flooding depths thus serves well to illustrate the distribution of ground elevations across this protected area. Elevations typically range from approximately +10 feet to -8 feet (MSL), with the higher elevation reaches located south of the Chef Menteur Highway. This Highway follows along a ridge of “high ground” known as the Bayou Sauvage ridge which is the result of an earlier river depositional channel (see Chapter 3), and this slight ridge serves to nearly separate the large northern section of the protected area from a smaller basin to the south. This separation was incomplete, however, as floodwaters managed to cross this ridge at a number of locations.

The New Orleans East protected area encompasses some of the lowest elevation lands in the greater New Orleans populated region, and the results of the full flooding of this protected basin were thus catastrophic, especially with regard to damage to homes and properties. As shown in Figure 2.12, loss of life was moderate (on the order of 120 persons, to date), however, largely because of the relatively effective pre-evacuation of this exposed

outlying area, and the relatively moderate rate at which the waters eventually filled the low-lying populous areas at the western end of this protected area. Because the area flooded and filled progressively over the course of the day on August 29, the storm surge subsided as it filled and the eventual filling extended only to approximately +2 feet (MSL) in the populous western end of the protected area; accordingly portions of the "high ground" along the southwest edge of the protected area remained above water (as shown in Figure 7.4.) The open, unpopulated eastern portion of the protected area initially filled to somewhat higher elevations, however, as it was relatively rapidly filled by the massive beaching and erosion of the New Orleans East back levees fronting the GIWW channel and Lake Borgne.

7.2 New Orleans East Hurricane Protection System

Figure 7.2 shows the results of a post-hurricane assessment of the condition of the primary levee system surrounding the protected area (IPET; March 10, 2006.) This protection system, which includes earthen levees, I-wall, T-wall, and sheet pile sections, was designed by the USACE as part of the Lake Pontchartrain and Vicinity Hurricane Protection Project. The NEO protected area also includes a secondary or "local" levee that separates the developed portions of the region from the wetlands to the east (Figure 7.1). The primary purpose of the secondary levee is interior drainage control rather than hurricane protection, and it was of lesser height than the main frontage levees (elevations typically on the order of +5 to +6 feet, MSL as opposed to elevations of +14 to +18 feet for the main perimeter frontage levees.)

The New Orleans East hurricane protection system is divided for planning and management purposes into individual segments, or "reaches," which are defined by physical characteristics, elevation, and/or potential consequences. For consistency, the names assigned to the individual reaches by the USACE will be used in this chapter. Figure 7.5 illustrates these section designations, and also indicates the locations of other points that will be discussed in this chapter.

The eastern edge of the protected area is defended by the New Orleans East Levee, an approximately 8.5 mile long earthen levee segment consisting largely earthen levees with 3 to 4 horizontal: 1 vertical side slopes, fronted on the outboard side by cypress swamps and wetlands. The southern boundary of the protected area (along the north bank of the east-west trending shared GIWW/MRGO channel) is defended by the New Orleans East Back Levee (to the east) and the adjacent Citrus Back Levee (to the west). These two reaches, which together measure approximately 18 miles in length, are largely comprised of earthen levee sections interspersed with sections comprised of concrete floodwalls atop lower height earthen levee sections and/or sheet pile wall segments. The IHNC East Levee is an approximately 3-mile reach primarily comprised of concrete floodwalls atop earthen levees. As its name implies, the portion of the levee system separates the western edge of the protected area from the adjacent IHNC. Continuing clockwise are the New Orleans Lakefront and Citrus Lakefront Levees, which include both earthen levees and composite concrete floodwall/earthen levee sections. Finally, the eastern 12.5 miles of the northern Lake Pontchartrain frontage is the New Orleans East Lakefront levee, and earthen levee with geometry similar to that of the adjoining New Orleans East Back Levee (just around the corner, along the eastern edge of the protected area.)

7.3 Performance of the New Orleans East Hurricane Protection System in Hurricane Katrina

7.3.1 Overview

Figures 7.1 and 7.2 show the locations of damage to the levee system surrounding the New Orleans East (NOE) protected area. The most significant damage to the system occurred to East Back Levee that fronts the GIWW and Lake Borgne. Here the storm surge completely destroyed (and massively eroded) large expanses of earthen levee in the southeastern corner of the NOE protected area. Additional smaller, but nevertheless significant breaches also occurred along other portions of these NOE back levee reaches. As the storm surge next passed west two significant levee breaches occurred, both due to overtopping, along the north bank of the east-west trending channel of the GIWW/MRGO. Damage (mostly in the form of scour) also occurred along the IHNC East Levee and portions of the New Orleans Lakefront Levee located near the Lakefront Airport as the storm surge raised the water levels within the IHNC. Finally, the reverse (counterclockwise) swirl of the storm winds raised the levels along the south shore of Lake Pontchartrain. Portions of the levee system fronting Lake Pontchartrain, such as the New Orleans Lakefront, Citrus Lakefront, and New Orleans East Lakefront Levees, generally performed well in the hurricane, as did most of the New Orleans East Levee located to the east.

7.3.2 Chronology of Events in the New Orleans East Protected Area

It is believed that water first entered the NOE protected area between about 5:00 a.m. to 5:45 a.m. on August 29 as a large section of earthen levee in the southeastern corner of the protected area catastrophically eroded and breached, as a result of wave action and possible seepage associated with the rising storm surge from Lake Borgne. The levee system at this location was so severely damaged that it ultimately did little, if anything, to impede the storm surge that later peaked at this location. Water entering the NOE protected area through this breach then crossed the adjacent wetlands before being channeled, initially, by the Bayou Sauvage ridge (high ground underlying Highway 90) to the west. Video footage (and eyewitnesses) recorded at the Entergy Power Utility Plant near the Michoud Canal show this inflowing water appearing to arrive from the east at approximately 6:15 a.m. Storm surge simulations by the IPET team (IPET Report 2, March 10, 2006) indicate relatively low water levels in the adjacent GIWW at the 6:00 a.m. hour, indicating that the water first arriving at the Entergy plant did not result from simple overtopping of the levees closely adjacent to this plant.

The storm surge then passed westward along the east-west trending GIWW/MRGO shared channel and produced levee damage and several smaller breaches on the north side of the channel. These breaches added to the water already flowing into the area through the major breaches in the southeast corner. The surge then continued westward reaching the GIWW's "T" intersection with the IHNC channel. The surge passed to the north (and south) along the IHNC, and damaged a number of sections along the IHNC frontage.

As the hurricane then passed northward to the east of New Orleans, the counterclockwise direction of the storm winds also produced a storm surge southward towards the shore of Lake Pontchartrain. The lake level rose, but largely stayed below the crests of most of the lakefront levees. The lake rose approximately to the tops of the lakefront levees at a number of locations, especially along the shoreline of New Orleans East, and there was modest overtopping (storm surge + wave splash-over) and some resulting erosion on the crests and inboard faces of some lakefront levee sections along the Lake frontage. However, there were no breaches in this area. Overtopping occurred over a section of floodwall near the west end of the New Orleans East protected area lakefront, where the floodwall was lower than the adjacent earthen levee sections. This, too, added to the flow into the New Orleans East protected area, which was now beginning to fill with water even as the original storm surges subsided. As shown in Figure 7.4, water depths ultimately approached 10 feet in area. Sadly, some of the deepest waters were in the NOE protected area's principal residential neighborhoods.

7.3.3 Damage to Levee System Frontages

The following sections summarize damage to the individual frontages of the levee system (Figures 7.1 and 7.2). For consistency, locations are referred to using the designations assigned by the USACE Task Force Guardian levee system rebuilding team. These names associated with each of the main levee sections are shown in Figure 7.5.

7.3.3.1 GIWW/Lake Borgne Frontage; the New Orleans East Back Levee

As shown in Figure 7.5, the New Orleans East back levee extends from the southeast corner of the NOE protected area west along the GIWW waterway, and it fronts both the GIWW channel and Lake Borgne as well. As noted earlier, the most severe damage to the NOE Levee System occurred along an approximately 5,300 foot long section of the New Orleans East Back levee, which is situated in the southeast corner of the protected area (Figures 7.1 and 7.2). The protection system at this location consists of earthen levee sloped at 4 horizontal: 1 vertical with a 10-foot wide crown.

This damage to this segment of the levee system was similar to that which occurred along the Mississippi River Gulf Outlet (MRGO) levees in St. Bernard Parish: entire sections were completely eroded leaving virtually no trace of the original earthen levee (Figures 7.1 and 7.2). Figure 7.6 shows typical erosion along the eastern end of this levee frontage; the levee embankment is entirely removed by erosion along much of this reach.

This NOE back levee frontage is a “sister” section to the MRGO levee frontage along the northeast edge of the St. Bernard/Lower Ninth Ward protected area that also suffered similarly catastrophic erosion along miles of its length (see Chapter 6, Section 6.2.) These two levee frontages share a number of unfortunate, deadly characteristics. Both sections were constructed in large part using materials from the excavation of the adjacent shipping channels (the MRGO and the GIWW, respectively), and as a result both were comprised largely of unacceptably highly erodeable soils; including large quantities of sands and lightweight shell sands. (Figure 7.3 shows the official material designations for the constructed perimeter levees surrounding the NOE protected area. All are nominally

compacted fills, except for the “hydraulic fill” section along the NOE back levee.) Both levee frontages directly fronted the swollen waters of “Lake” Borgne (which is actually a bay, being directly connected to the open Gulf of Mexico), and so both sections experienced storm waves driven by winds that passed across large open distances; waves that gathered significant energy. Both sections had little or no effective protection on the outboard side from swamps or cypress groves, or other vegetation, etc., that could reduce the intensity of these waves. And both sections appear to have failed catastrophically, and eroded massively, producing massive breaches along thousands of feet through which passed a majority of the floodwaters that so catastrophically devastated the St. Bernard/Lower Ninth Ward and the NOE protected areas.

As described previously in Chapter 6, it is the conclusion of our ILIT investigation that the MRGO frontage levees likely failed, and suffered significant breaching, well before they experienced significant overtopping. The discussion of potential erosion mechanisms presented in Section 6.2 is applicable again here, and is worth revisiting on the part of the reader.

Whereas our investigation concluded that the MRGO frontage levees were apparently compromised before they were significantly overtopped, with the “sister” levees along the NOE back levee frontage it can be conclusively demonstrated that massive failures occurred prior to overtopping.

Figure 7.7 shows hydrographs of calculated (modeled, back-calculated) water levels vs. time during and after hurricane Katrina’s passage, as calculated by IPET, for locations at and near the NOE back levee frontage. Similar calculations by Team Louisiana give similar results. The storm surge at the western end of Lake Borgne rose fairly slowly to Elev. +4 feet (MSL), then as the eye of the storm approached more closely it rose rapidly and peaked at about Elev. +16 to +18 at about 8:30 a.m.(CDT; local New Orleans time.) After peaking, the storm surge dropped rapidly at this location. [Many of the hydrographs in this report, and others, are based on GMT (Greenwich Mean Time), and so must be converted to CDT (local time). Similarly, the hydrographs of Figures 7.7 and 7.9 are based on the NGVD datum, and actual MSL elevations are approximately 1.7 feet lower. Some adjustment to elevations as shown are being inferred herein, as the calculated elevations of Figures 7.7 and 7.9 may be a bit low (on the order of about a foot or so) based on field observations and similar calculations by Team Louisiana.]

Figure 7.8 shows calculated maximum storm surge (and also storm surge + wave) elevations, again based on IPET analyses, and also levee crest heights along this frontage. This figure shows that peak surge + waves might have overtopped this frontage at several locations at the eastern end, and at the far west end as the GIWW and MRGO “funnel” necks down to become the joint, east-west trending shared GIWW/MRGO channel.

There is well established evidence, however, that significant breaching had already occurred between about 5:00 a.m. to 6:00 a.m. Eyewitnesses, and a hand held video, clearly show that significant floodwaters approached from the east and arrived at the Entergy power plant located along the north side of the GIWW/MRGO waterway at 6:15 a.m., and that the depth of water increased rapidly over the next few minutes (indicating a large source.)

Figure 7.9 (top) shows the location of this power plant. There are only three possible breaches/sites that could have been the source of these well-timed floodwaters; (1) overtopping, and two breaches, along the Citris levees (along the GIWW/MRGO channel, to the west, (2) local overtopping adjacent to the power plant itself, and (3) the massive breaches at the southeast corner of NOE, along the NOE back levees fronting Lake Borgne. Given the crest heights, and water elevations vs. time, it can be established that the overtopping required for options (1) and (2) above did not begin until well after 7:00 a.m., so the only likely source of these floodwaters appears to be the massively eroded sections of the NOE back levee frontage.

Floodwaters from these breaches would have been channeled by the Bayou Sauvage ridge (high ground underlying Highway 90), and would have come west around the top of the Michoud Canal to the Entergy power plant fairly rapidly. Allowing for the distances involved, there must have been significant breaching and inflow by at least 6:00 a.m., and likely earlier. Water levels along this frontage would only have been on the order of Elev. +8 to +10 feet (MSL) by 6:00 a.m., and would not have passed over (even with wave run-up) the levees along this frontage (with crest elevations of +15.5 to +19 feet, MSL.) Accordingly, it appears that significant levee failures, and breaching, occurred prior to significant overtopping.

Like the MRGO frontage levees discussed in Section 6.2, this catastrophic failure was due primarily to the use of inappropriate, highly erodeable levee embankment fill materials, including sands and lightweight shell-sands. As discussed in Section 6.2, the actual mechanisms of erosion that led to this failure are likely to have included wave scour on the outboard sides, wave run-up and resulting notching and crenellation of the levee crests, exploitation of this by splashover overtopping, and through-flow erosion (which would have, initially, been most pronounced low on the back or protected side of the levees.) These mechanisms, working alone or in combination, appear to have compromised the earthen levees well before the storm surge peaked, and therefore, well before the levees were overtopped in the conventional sense of the word.

Damage to the NOE back levee reach also occurred further west, between the interior secondary levee and the Michoud Canal. A sheetpile levee “transition” section located near Pump Station 15 deflected and tilted inward (i.e., toward the protected side, see Figure 7.10), as the result of overtopping-induced erosion at the base of the backside of the sheetpile wall. Sheet piling was used at these locations to transition between concrete floodwall and full-height earthen levee sections. The tops of the damage sheet pile wall had pre-Katrina elevations that were less than the immediately adjacent concrete floodwall sections, and hence scour at this location was worsened by preferential overtopping during the peak of the storm surge. Further to the west near the Air Products Corporation site, a similar sheet pile transition section overturned and collapsed in response to scour and the associated loss of passive resistance on the protected side (Figures 7.12 and 7.13). Once again, the top of the damaged section was at a lower elevation than adjacent levee segments resulting in highly concentrated overflow (and resulting scour, that laterally unbraced the sheetpile wall) at this location. Note that there is little or no evidence of overtopping erosion adjacent to the failed sheetpile transition section. This is one of numerous cases wherein the adjacent long reaches

of full-height earthen levee and concrete floodwall-topped levee both performed well, but where inadequate attention was paid to effecting a safe “transition” between these two major project elements; a tragic failure of attention to detail, and an adverse product of the piecemeal process by which these massive and complex levee systems are constructed in individual segments and stages.

7.3.3.2 The Michoud Area and the Citrus Back Levee

The Michoud area levee systems site extends along the GIWW from Michoud Slip to (and around) the Michoud Canal. The site is located below and immediately west of the Interstate 510/Highway 47 bridge near the Entergy New Orleans Corporation's power plant. Scour was noted at the base of the rear side of the concrete floodwalls surrounding both Michoud Slip and Michoud Canal; however, breaching did not occur at this location and overall system performance was good (Figure. 7.11). In addition to the video of early morning flooding here highlighted earlier, mounted security cameras later captured dramatic images of levee overtopping during the peak of the storm surge (see Figures 7.17 and 7.18.)

West of the Michoud sector, the remainder of the levee reaches along the north bank of the GIWW/MRGO channel constitute the main Citrus back levee section. As the risen waters of Lake Borgne were pushed west along the shared GIWW/MRGO channel, overtopping occurred along considerable lengths of the Citrus back levee frontage. Many earthen embankment sections sustained this overtopping with little or no damage, while adjacent sections suffered variable amounts of overtopping-induced erosion on their back (inboard side) slopes, but without full breaching.

A major failure did occur along this frontage, at the Citrus back levee floodwall. This site is located in the industrial corridor south of Chef Menteur Highway along the GIWW. Because its protection system consists of a relatively short floodwall segment situated between longer stretches of full-height earthen levee, the site provides a unique opportunity to compare the performance of different types of levees subjected to identical storm surge loadings. The levee system at the site principally consists of an approximately 3000 foot long I-wall with a short (~ 80 feet) T-wall section, and a 50-foot long T-section with a steel gate. The adjacent earthen levee sections are sloped at 4 horizontal: 1 vertical and include a 10 foot wide crown. The I-wall tilted and deflected significantly in response to the rising storm surge. Deflection along the 3000-foot length of the concrete I-wall section from severe (i.e., almost completely tilted over, Figure 7.15) to moderate (i.e., lateral movement of several feet, with limited tilting, Figure 7.14). Deflections were generally greater near the eastern and middle segments of the floodwall.

Scour trenches developed along the full length of the floodwall on the protected side, as overtopping cascaded over the tops of the floodwalls. In many instances, these trenches were located several feet from the base of the wall (indicating progressive tilting of the floodwalls, and thus the waters falling farther to the inboard side) and some had widths of 7 feet or more. A massive scour hole was found behind to the most tilted segment of the I-wall system. Localized scour was also noted at the western edge of the I-wall where it connects to the earthen levee, representing yet another example of an inadequate “transition” detail connecting two disparate sections. These scour-induced trenches reduced the lateral support

for the sheetpiles and the concrete floodwall they supported, and the lateral forces of the outboard side storm surge pushed the laterally unbraced floodwalls sideways. Figure 7.14 shows the eroded trench at the inboard side of a floodwall section that experienced only limited movement; note the heave of soils immediately at the toe of the sheetpiles/floodwall. Figure 7.15 shows a view of the outboard side of a floodwall section that was nearly completely overturned. In this figure, the “gap” between the sheetpiles and the non-displaced outboard side levee embankment toe can be clearly seen. As discussed in numerous other sections of this report, the formation of this water-filled gap served to increase the lateral forces acting against the outboard side of the sheetpile/floodwall.

Post-event topographic maps of the area show a localized low area close to the large scour hole. The tilting of the wall effectively reduced its top elevation, which is likely to have attracted additional overtopping at this location, causing localized erosion that ultimately developed into the large scour hole. This may have, in turn, further exacerbated tilting of the floodwall due to loss of passive soil resistance. It is worth noting that damage to the levee system at this location was almost entirely limited to the relatively short floodwall segment. The adjacent earthen levee segments performed well despite having been subjected to an identical storm surge loading.

As noted above, the floodwall protection system included two isolated segments which were T-wall segments, both of which performed well (i.e., little if any permanent deflection) despite the scour that occurred along their bases. This suggests that the increases lateral and rotational stability and stiffness provided by the battered structural piles supporting these T-wall sections were very useful at this location.

The earthen levee sections east and west of the floodwalls also performed well (i.e., no breaching or significant distress), though at some sections, particularly to the east of the floodwalls, isolated scour holes developed along the levee slopes on the protected side. One of the worst of these is shown in Figure 7.16. The soil exposed in these scours indicated the levees were comprised of largely cohesive materials, and this likely explains their favorable performance with regard to successfully resisting erosion and full breaching (failure) during sustained overtopping.

Figure 7.17 shows a still image from a security videotape showing significant overtopping of the earthen levee adjacent to the Entergy power plant, immediately east of the highway bridge to the St. Bernard parish. Figure 7.18 shows the same site after the hurricane had passed. The overtopping had produced moderate damage, but again no beaching of the levee crest and no failure at this location. Erosion-related performance was generally more favorable than these two examples along the earthen levees that comprised most of the Citrus levee frontage, and many sections showed no indication of overtopping erosion whatsoever.

7.3.3.3 The IHNC Frontage (IHNC East Levee)

The levee system located along the IHNC is primarily comprised of conventional floodwall-topped levee sections interspersed with a number of gate and transition structures. Overtopping occurred along almost all of this levee frontage. Overall performance was good along most of this frontage, with only one major breach at the extreme north end of this reach.

There were also, however, numerous partially evolved erosional problems at “transitions” along this frontage, and some of these might have been more serious if the inboard side had not already been filling with water from breaches at other locations.

Figure 7.19 shows a typical example of overtopping-induced scour behind a concrete floodwall along this frontage. This was common along this frontage, but no full failures resulted. It is not possible to know with certainty to what extent this type of erosional damage was limited by the fact that waters were likely already accumulating at the inboard sides of these floodwalls due to overtopping and breaches at other locations.

Figures 7.20 through 7.22 show several examples of the 8 locations along this frontage where erosion occurred, but did not develop fully to the point of “failure”, at transitions between adjoining flood system elements. Transitions between full height earthen levees and adjacent, composite levee/floodwall sections, and transitions between levees and concrete gate structures (with rolling steel floodgates), were routinely problematic in this regard, and it was common to find partially developed erosion problems at both ends of most gate structures along this frontage. Inadequate attention to transition details, especially to lateral embedment of transitions, and differences in top elevations of adjoining elements, were common. Also disconcerting were sites where the eroded materials appeared to be comprised, at least in part, of lightweight shell-sands; materials notorious for lack of erosion resistance that have no place in these levees protecting large populations.

At all locations, these “transition” erosional features were partially developed, and so no full failures developed. This initially puzzled our field teams, until we learned that floodwaters had been already rising on the inboard (protected) side of levees and floodwalls while the overtopping was occurring; effectively reducing the gradient across these erosional features and minimizing the progression of the erosion. These are features that warrant significant additional attention during reconstruction, as these features might otherwise prove far more dangerous in future events if the inboard side is not already flooding.

At the north end of the IHNC frontage, at the corner where it joins the Lakefront levees, a full breach did occur. This was a complex “transition” section where three utilities consisting of (1) a major highway (the I-10), (2) an adjacent active railroad line, and (3) a surface roadway between these two, all cross the federal perimeter levees. This transition is rendered even more complex by the fact that it is the “corner” of the NOE protected area.

Figure 7.23 shows this location in plan view. Significant overtopping occurred along a nearly mile-long section of the Lakefront levee that had an unexpectedly low floodwall crest height, and this flow passed through the gravel ballast of the railroad embankment (a local low spot, as it was pervious) and eroded the adjacent earthen perimeter levee. This flow also eroded the transition between a concrete floodwall and the adjoining earthen levee section beneath the elevated highway, as shown in Figure 7.24.

7.3.3.4 The New Orleans Lakefront and Citrus Lakefront Levee Frontages

The lakefront levee systems include both earthen levees and composite levee/floodwall sections. With one exception, these performed well. This exception was a

nearly mile-long section of floodwall at the west end, behind the Old Lakefront Airport. This section had a unexpectedly low floodwall crest elevation, and it experienced significant localized overtopping, and resultant scour at the inboard side toe of the concrete floodwall, as shown in Figure 7.26. This overtopping-induced scour did not produce a failure, however, so the overtopping flow simply added to the misery of an area that was already flooding as a result of numerous failures that had already occurred to the south.

Only modest damage, primarily in the form of scour, occurred along the remainder of the Lake Pontchartrain frontage. The levee system along this reach was comprised of both floodwall and conventional earthen sections. Storm surge simulations indicate that the lake levels were close to but not greater than the top of the levees, and therefore the scour most likely resulting from wave splash over rather than sustained sheetflow overtopping. Figure 7.27 shows one of the few locations where minor repairs had to be made for erosion. Figure 7.28 shows a second location where limited overtopping produced minor erosional damage. Overall, the performance of levees along the Lakefront, east of the Old Lakefront Airport, was very good.

7.3.3.5 The New Orleans East Levee Frontage

Similar performance was also noted along the eastern levee frontage, which is buffered from the nearby lake systems by a large stretch of wetlands to the east. Figure 7.29 shows a post-event view of a typical levee segment along this frontage. No damage at all was noted along most of this frontage, and only limited erosion at a few locations. This was despite evidence suggesting that overtopping had occurred along at least some portions of this frontage. This favorable performance was likely due to: (1) the use of compacted, clayey fill for the levee embankments (materials with a high resistance to erosion), and (2) the presence of significant widths of swamps and cypress and other vegetation on the outboard sides of the levee (which served to buffer the wave action.)

The only notable damage that occurred in this area was scour in a floodwall-earthen levee transition section that was part of a railroad gate structure. This produced a minor “breach”, but given the massive flows that were admitted through the catastrophically eroded lengths of the New Orleans East back levee immediately to the south, this was a relatively unimportant feature in this event. It does, however, provide yet another example of problems with handling of “transitions”, and the site should be re-assessed and mitigated as it might represent a more serious potential vulnerable point in future events if the inboard side lands are not already rapidly filling with floodwaters.

7.4 Summary of Findings for the New Orleans East Protected Area

The key findings of this chapter may be summarized as follows:

- The catastrophic breaching of the New Orleans East Bask Levee System in the southeast corner of the polder was responsible for much of the flooding of the New Orleans East protected area. While there is limited data as to the exact time that the breach developed, the available evidence strongly suggests this occurred well in advance of the peak of the storm surge. This implies that the levee at this location

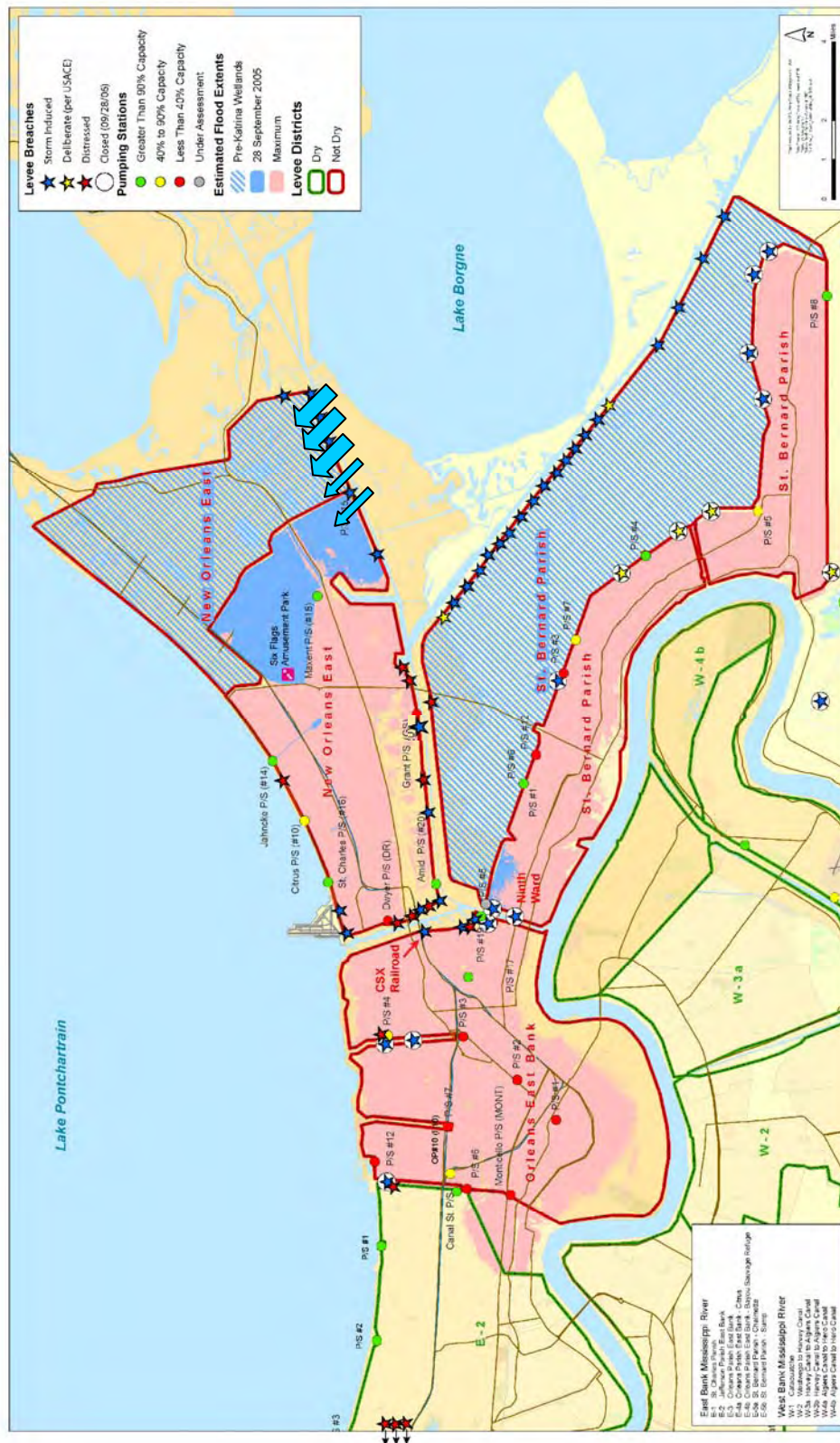
failed not in response to simple overtopping, but rather as a result of wave action and/or through-seepage erosion, and this levee frontage appears to have been significantly compromised, related to the rising water levels in the GIWW. The use of fill materials known to be highly erodeable, from the excavation of the adjacent GIWW shipping channel, resulted in short-term cost savings that are, in hindsight, difficult to justify against the massive damages and the loss of life engendered by the catastrophic erosion and failure of these levees.

- With the notable exception of the levee system in the southeast corner, the conventional full-height earthen levees that protect most of the New Orleans East protected area performed quite well. This is despite, in some cases, significant overtopping that occurred during the peak of the storm surge.
- The performance of concrete floodwalls was uneven. In some cases these systems performed well even when overtopped (e.g. along the IHNC frontage). In other situations (e.g. collapsed Citrus Back Levee Floodwall) the performance was unsatisfactory.
- Levee transition sections and gate structures were routinely problematic. Common problems, often because of the differences in elevation between adjacent sections, which resulted in concentrated or preferential overtopping. In many instances, damage also occurred at these locations because of the contrast in erosion resistance between adjoining sections (e.g. flood wall-earthen levee transitions).

7.5 References

Interagency Performance Evaluation Task Force, (2006), "Performance Evaluation, Status and Interim Results, Report 2 of a Series, Performance Evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System," March 10, 2006.

IPET, (2006), "Performance Evaluation of the New Orleans and Southwest Louisiana Hurricane Protection System, Draft Final Report of the Interagency Performance Evaluation Task Force, Volume VI – The Performance – Interior Drainage and Pumping," available online: <https://ipet.wes.army.mil/>, date accessed: June 1, 2006.



Source: Modified after USACE, 2005

Figure 7.1: Map showing principal features of the main flood protection rings or “protected areas” in the New Orleans area.

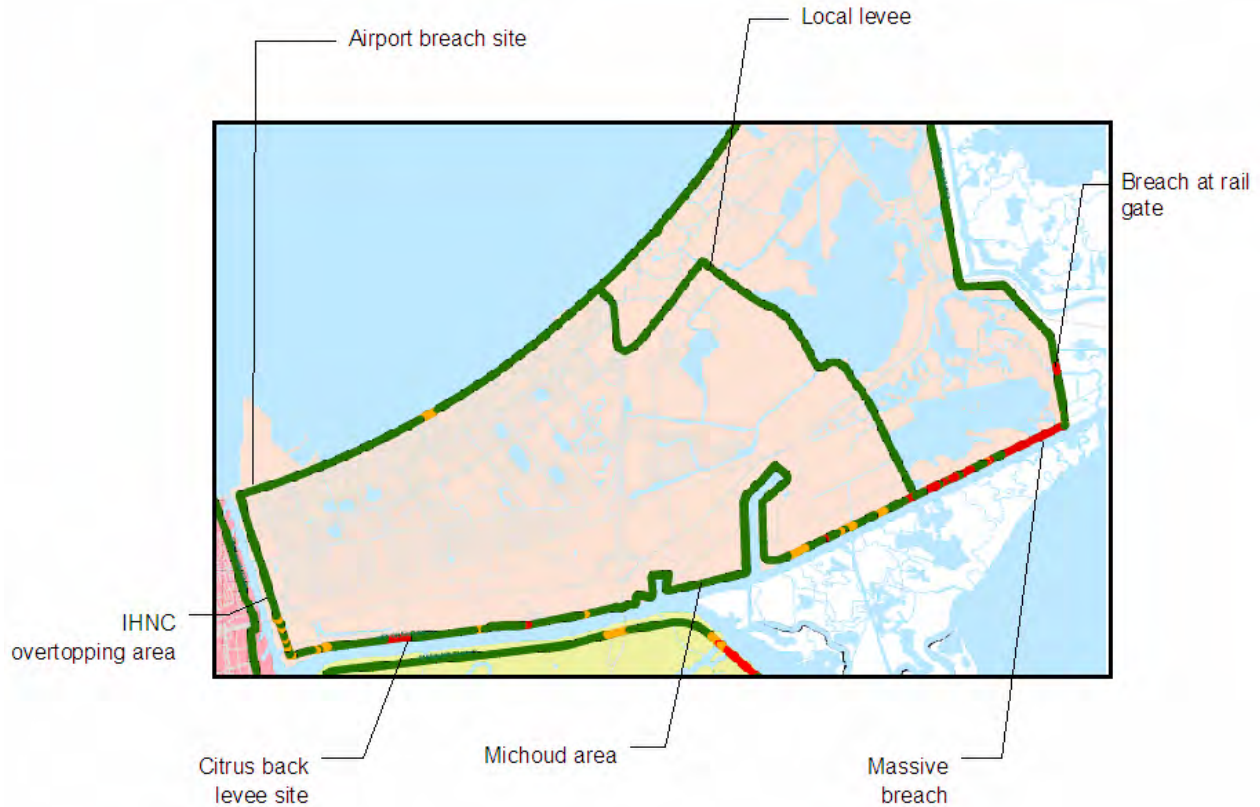
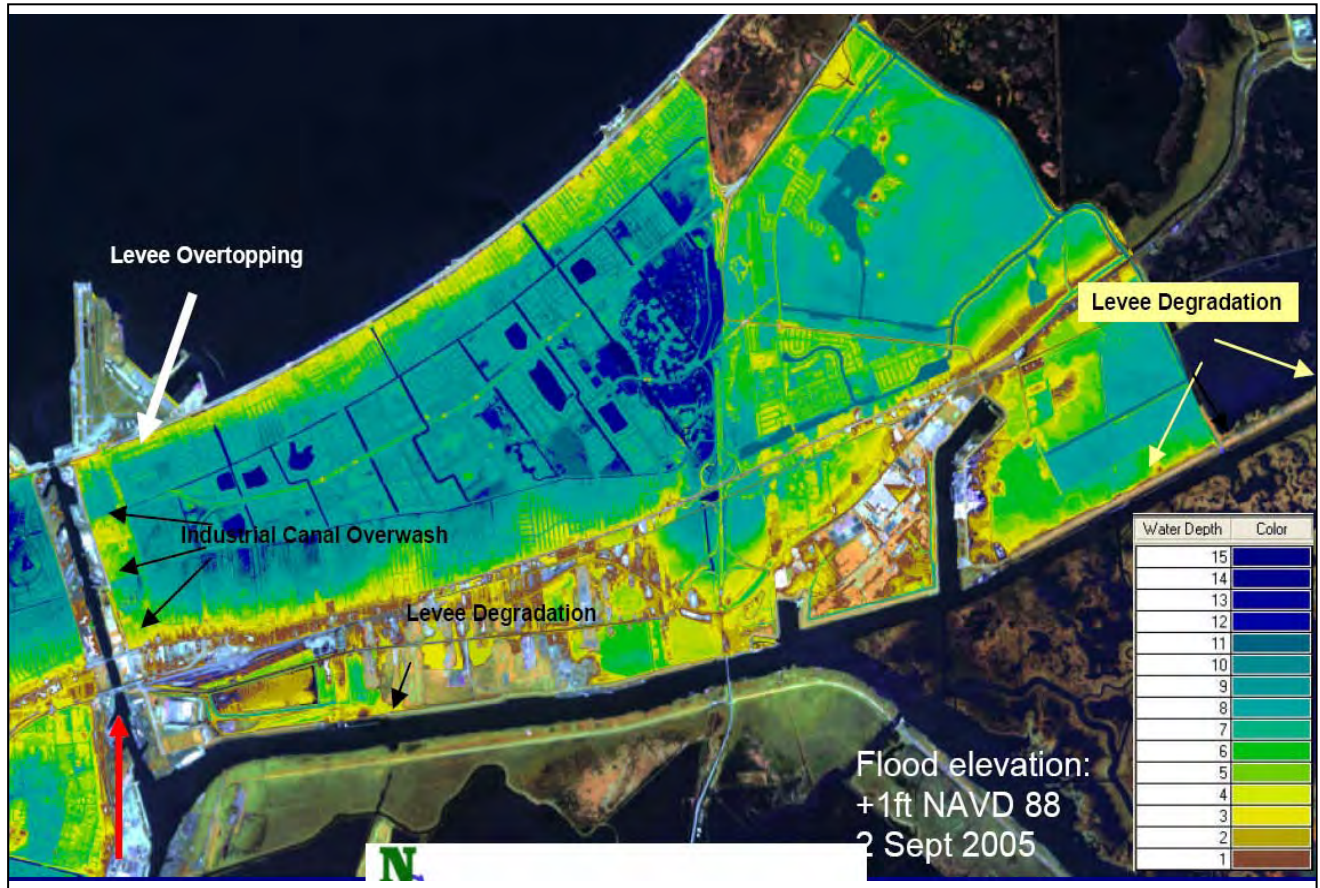


Figure 7.2: Damage locations in the NOE protected area (base map from USACE.) Color indicates severity of damage, with red being the worst. [IPET; March 10, 2006]



Figure 7.3: Construction materials and methods, New Orleans East. [IPET; June 1, 2006]



Source: LSU Hurricane Center, 2006

Figure 7.4: Depth of flooding of New Orleans East on September 2nd (4 days after Hurricane Katrina)



Figure 7.5: Principal sections of the New Orleans East perimeter defense levees; including the Lakefront Levees, the New Orleans East Levee, the New Orleans East Back Levee, the Michoud Canal, the Citrus Back Levee, and the IHNC Levees.



Photo courtesy of USACE

Figure 7.6: Some of the most severe damage to the New Orleans regional levee system occurred along this section of the New Orleans East Back levee, which is situated in the southeast corner of the protected area, facing south toward Lake Borgne.

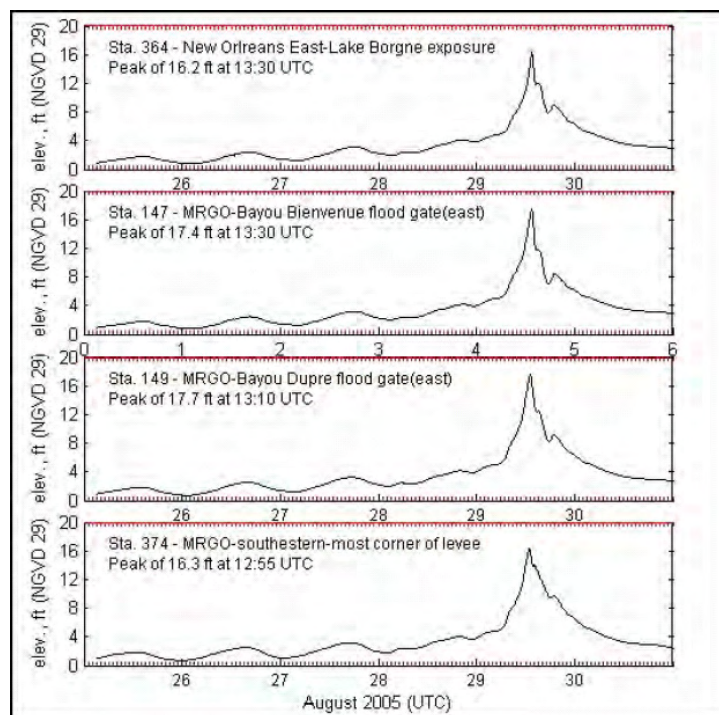


Figure 7.7: Approximate hydrograph of storm surge elevation (feet, MSL) vs. time at the west end of Lake Borgne. [IPET Interim Report; April, 2006]

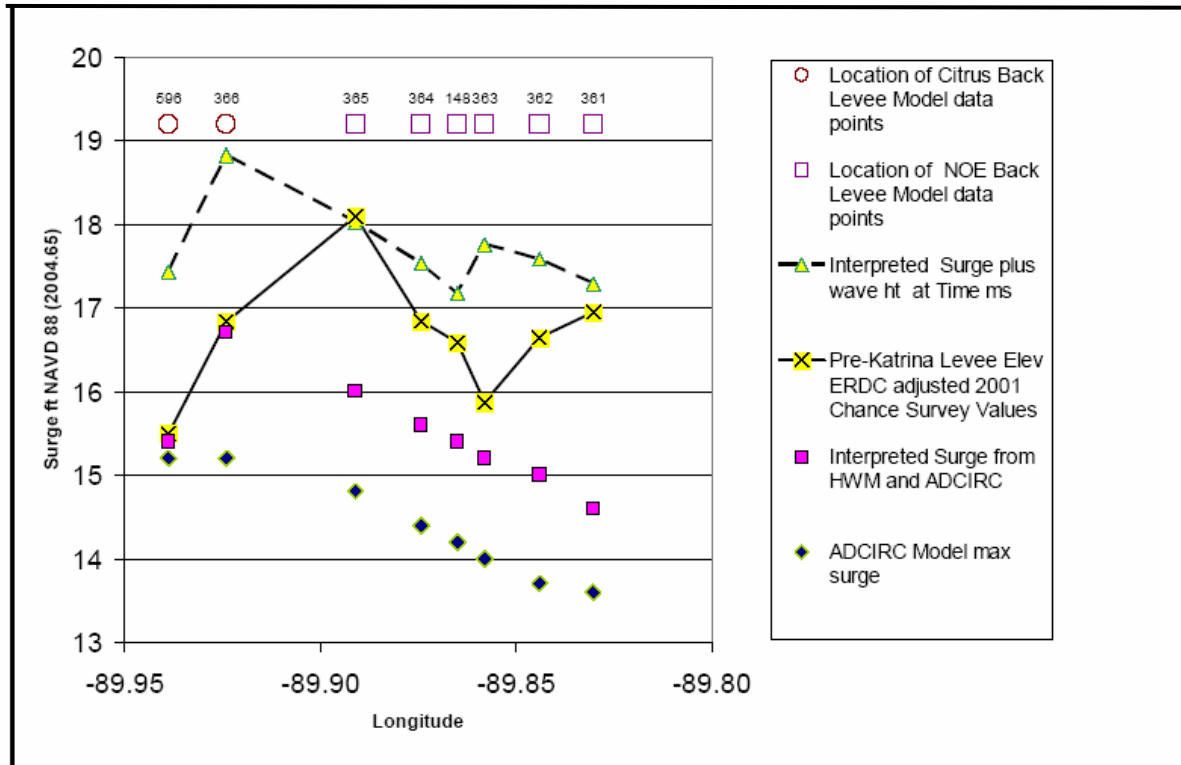


Figure 7.8: Pre-Katrina crest elevations, and various estimates of storm surge + wave height; New Orleans East back levee facing Lake Borgne [IPET; June 1, 2006]

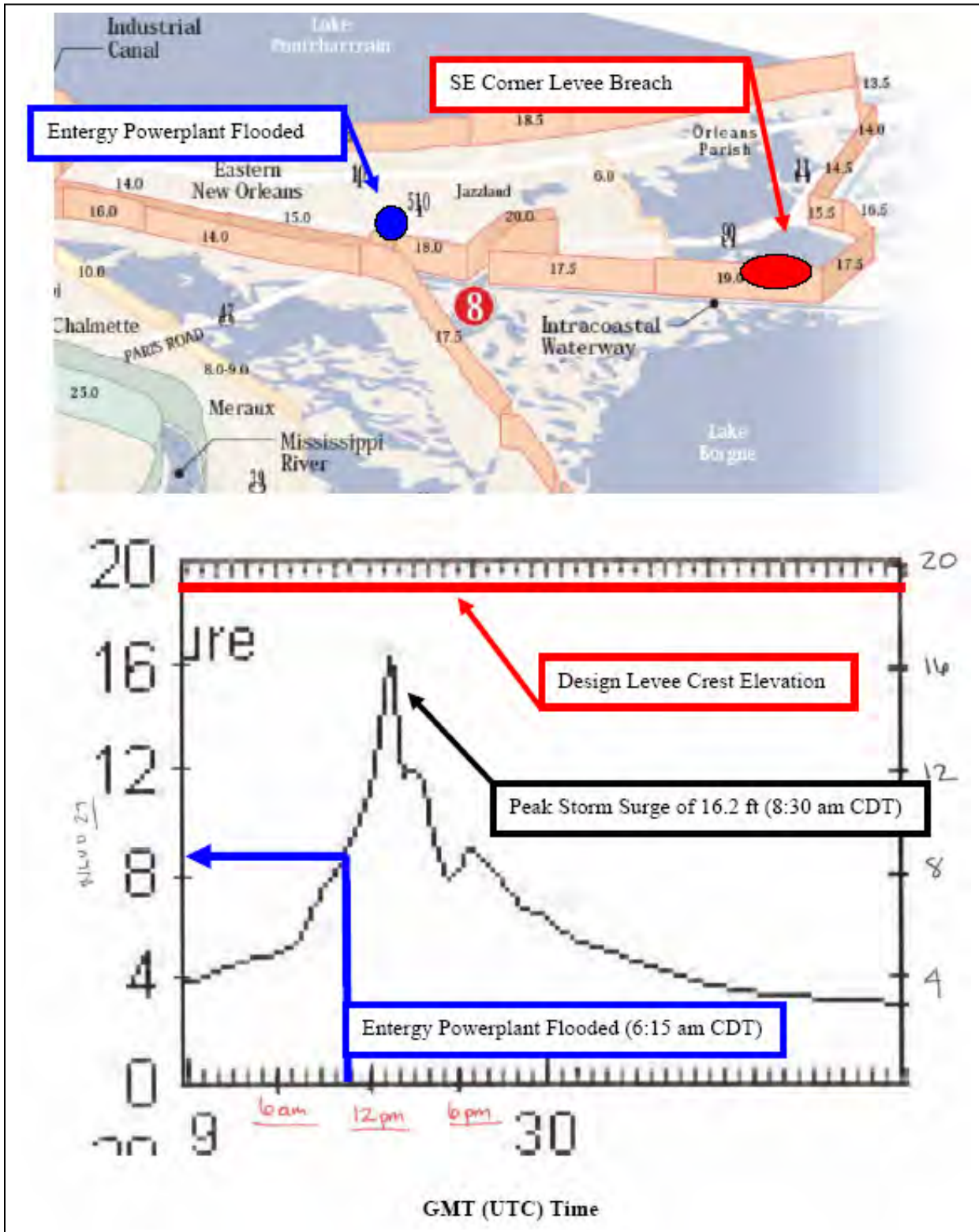


Figure 7.9: Timing of observed flooding at Entergy Powerplant and storm surge at New Orleans East back levee breach (southeast corner fronting Lake Borgne.) [Times shown are UTC or Greenwich Mean Time. Elevations shown are in feet, NGVD29.]



Photograph by J. Wartman

Figure 7.10: Deflected and tilted sheet pile sections near Pump Station 15.



Photograph by J. Wartman

Figure 7.11: Scour at the base of floodwalls near the Michoud Canal.



Photo by Dr. Les Harder

Figure 7.12: Failed sheetpile transition at the Air Products Corporation site; NOE back levee.



Photo courtesy of USACE

Figure 7.13: Second view of failed sheetpile transition.



Photograph by J. Wartman

Figure 7.14: Significant lateral deflection of the Citrus Back Levee floodwall, seen from the inboard (protected) side. Note the heave adjacent to the displaced sheetpiles and wall.



Photograph by J. Wartman

Figure 7.15: Deflection and tilting of another section of the Citrus Back Levee Floodwall, this time viewed from the outboard side. Note the gap between the outboard levee toe section and the sheetpile curtain.



Photograph by J. Wartman

Figure 7.16: Scour varied greatly along the Citrus Back Levee. It was significant on the back (inboard side) slope of the levee at this location; nearly breaching the levee crest.



Sill photo from security video at Entergy Powerplant

Figure 7.17: Still image from security videotape taken at Entergy power plant showing overtopping adjacent to the I-510/Hwy 47 Bridge on the NOE Back Levee.



Photograph by Rune Storesund

Figure 7.18: Post-Katrina photo of the same levee section shown above in Figure 7.17.



Photograph by J. Wartman

Figure 7.19: Minor scour along the base of the IHNC floodwall. Note the boat pushed against the outboard (flood) side of the wall.



Photograph by Rune Storesund

Figure 7.20: One of numerous examples of partially exploited erosive vulnerability at a “transition” section along the IHNC levee frontage; in this case a transition from a gated concrete floodwall to a full height earthen levee section.



Photograph by Francisco Silva-Tulla

Figure 7.21: Another example of partially exploited erosive vulnerability at a “transition” section along the IHNC levee frontage; in this case a transition from a roadway floodgate to a full height earthen levee section.



Photograph by Rune Storesund

Figure 7.22: Erosion at the east bank IHNC CSX Rail Crossing.

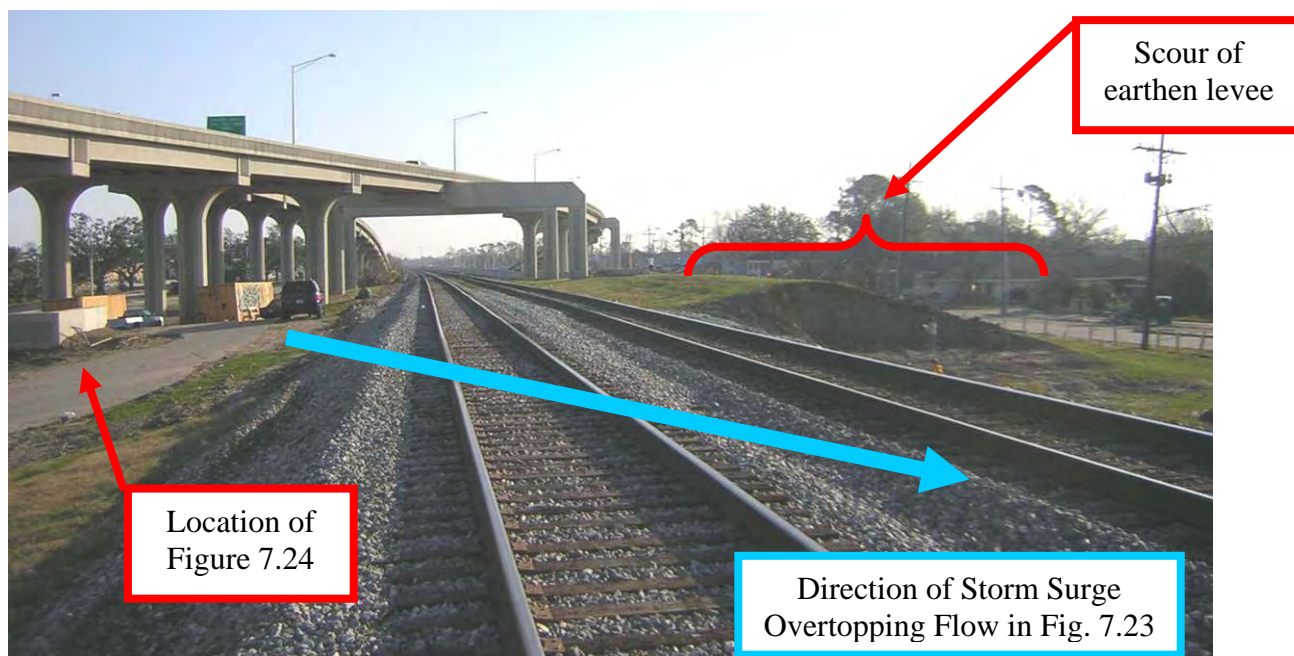


Figure 7.23: Storm-surge induced overtopping traveled through the granular gravel ballast for the railroad line and eroded the railroad line embankment, which served as a transition levee between the concrete floodwall and the earthen levee shown in Figure 7.25. [Base image from Google Earth, 2006]



Photograph by Rune Storesund

Figure 7.24: IHNC levee near the Lakefront Airport adjacent to the railroad section from Figure 7.23, showing erosional failure and scour at transition to concrete floodwall protecting highway support.



Photograph by Rune Storesund

Figure 7.25: Significant erosion was observed on the levee adjacent to (and behind) the floodwall shown in Figure 7.24. The storm surge overtopped the floodwall and railroad ballast and failed the earthen levee behind the railroad.



Photograph by J. Wartman

Figure 7.26: Scour near the base of a floodwall near the Lakefront Airport.



Photograph by Rune Storesund

Figure 7.27: Lakefront levee near the Jahncke Pump Station outfall structure, where minor overtopping erosion occurred. These levees performed well and only minor, surficial damage was observed.



Photograph by Rune Storesund

Figure 7.28: Observed scour at the Jahncke Pump Station outfall structure, Lakefront. Scour was limited to areas of soil-structure interfaces, and no full breach occurred.



Photograph by Rune Storesund

Figure 7.29: Condition of levees east of HWY 11 (location 3 on Figure 10.6) in October 2005. These levees performed exceptionally well and were not eroded during Hurricanes Katrina or Rita.

CHAPTER EIGHT: THE ORLEANS EAST BANK (DOWNTOWN) AND CANAL DISTRICT PROTECTED AREA

8.1 Overview

The most populous of the four major protected areas that suffered significant flooding during Hurricane Katrina was the Orleans East Bank (downtown) protected area. As shown in Figures 2.4, 8.1 and 8.2, the Orleans East Bank (downtown) section is one contiguously protected section. This protected unit contains the downtown district, the French Quarter, the Garden District, and the “Canal” District. The northern edge of this protected area is fronted by Lake Pontchartrain on the north, and the Mississippi River passes along its southern edge. The Inner Harbor Navigation Canal (also locally known as “the Industrial Canal”) passes along the east flank of this protected section, separating the Orleans East Bank protected section from New Orleans East (to the northeast) and from the Lower Ninth Ward and St. Bernard Parish (directly to the east.) Three large drainage canals extend into the Orleans East Bank protected section from Lake Pontchartrain to the north, for the purpose of conveying water pumped north into the lake by large pump stations within the city. These canals, from west to east, are the 17th Street Canal, the Orleans Canal, and the London Avenue Canal.

Figure 8.2 shows how this single, contiguously protected unit can be sub-divided into several localized sub-basins separated by a series of ridges, levees and canals. The base map of Figure 8.2 is the flooding map of Figure 2.17 (repeated here) which shows the flooding on September 2, four days after the passage of Katrina. The elevation of the top of the floodwaters in this figure is Elev. +3 feet (NAVD 88). This is approximately the peak flooding, and the depths of flooding shown at this point in time reflect the underlying basin topography and thus serve well to illustrate how the overall protected zone can be approximately subdivided into four separate zones or sub-basins.

The original city of New Orleans had been founded on the high ground adjacent to the Mississippi River (along the southern edge of this protected area.) The river “climbs” within its own channel, periodically depositing overbank sediment deposits which form “natural levees” to constrain its path, until it rises above the surrounding countryside. Then, periodically, the river breaks through its own “natural levees” and takes a new path to the Gulf. The riverbank deposits thus represent the highest ground locally, and it was here that the city began.

As shown in Figure 8.2, this high ground adjacent to the river now comprises much of the expensive Garden District, and much of downtown New Orleans and the historic French Quarter as well. Due to their elevation (typically Elev. +2 feet above Mean Sea Level and higher) these areas remained largely unflooded. Most of the remainder of this large and densely populated protected zone lies at lower elevations, however, and so most of the rest of this zone was flooded.

As also shown in Figure 8.2, a ridge of high ground known as Metairie Ridge separates the low-lying northern (Canal District) portion of this protected area from the

southern half. Metairie Ridge is the result of a previous river “stand”, and resulting river deposits. The Metairie Ridge did not quite successfully separate the northern and southern halves of this protected section during Katrina; flow passed over the ridge at a number of locations carrying floodwaters from the catastrophic northern drainage canal breaches into much of the rest of the protected area to the south. This flow over (across) the Metairie Ridge was noted by eyewitnesses (Van Heerden, 2006), and is also confirmed by calculations of flows through the various breaches.

As described previously in Chapter 2, the initial breaches in this protected area occurred along the eastern flank (on the west bank of the IHNC). Several breaches occurred along this frontage. These breaches allowed significant amounts of water to flow into the adjacent neighborhoods, but these breaches were non-catastrophic; they breaches did not scour to a depth below mean sea level, so that as the storm surge subsequently subsided the flow inwards through these breaches was eventually halted as the IHNC water levels fell below the (mean sea level plus) “lips” of these breaches. Our current estimates, based on simplistic calculations of flow and surge heights vs. time, suggest that approximately 10% to 20% of the eventual flow into the overall Orleans East Bank (Downtown) protected area came through these breaches. Similar calculations, in a bit more detail, by Team Louisiana suggest that approximately 12 to 15% of the overall floodwaters eventually filling the Orleans East Bank protected area came through the breaches on this east bank of the IHNC (Mashriqui, 2006).

The vast majority of the flow into the Orleans East Bank came through the three subsequent, catastrophic breaches in the drainage canals at the northern edge of the Orleans East Bank protected area. As shown in Figures 8.1 and 8.2, one catastrophic breach occurred on the 17th Street drainage canal and two catastrophic breaches occurred on the London Avenue drainage canal. These all eroded (scoured) to depths well below mean sea level, and so continued to admit flow into the city from Lake Pontchartrain well after the initial storm surge had subsided. The drainage canal located between these two (the Orleans Canal) did not suffer any breaches, but the southern end of this canal was unfinished and a “gap” (low area) in the floodwall at the southern end of this canal allowed water to flow freely into New Orleans for a number of hours during the peak of the storm surge.

It was, however, mainly the flow through the three catastrophic breaches in the 17th Street and London Avenue drainage canals that accounted for approximately 85% of the flooding that slowly filled this Orleans East Bank protected area during and after the storm. Flow from the canals overfilled the northern basin and eventually also flowed over the Metairie Ridge and into the other zones shown as flooded in Figures 8.1 and 8.2. This flooding continued to progress after the initial storm surge had subsided, and flooding in the southern portions of this protected zone continued to worsen overnight and into the three days that followed, finally equilibrating with the slightly inflated water levels in Lake Pontchartrain on Thursday (September 1.)

As discussed in Chapter 2, this flooding had catastrophic consequences, accounting for approximately half of the total loss of life in this event, and a similar share of the economic damages as well. The performance of the flood protection system in this Orleans East Bank

(downtown) basin is thus of great importance, and was studied in some detail by this investigation.

8.2 Performance of the Flood Protection System Along the West Bank of the Inner Harbor Navigation Channel (IHNC)

8.2.1 An Early Breach at About 4:45 a.m.

As described previously in Chapter 2, the first levee breach and failure in the metropolitan New Orleans area appears to have occurred along one of the banks of the IHNC.

Figure 8.3 shows water elevations at three gage stations as well as at a manual water elevation station in the IHNC as the hurricane storm surge initially began to raise the water levels throughout the IHNC region on the morning of August 29th. As the storm began to approach the coast, water levels within the IHNC began to rise. By about 4:30 a.m. the water level within the IHNC had risen to approximately +9 to +9.5 feet (MSL). Then, at approximately 4:45 a.m., two of the gauges near the Highway I-10 bridge registered a sudden change in the otherwise relatively constant rate of rise in water levels. The U.S. Geological Survey (USGS) gage at this location shows a precipitous drop in water levels at approximately 4:45 a.m. The Orleans Levee District gage was “sampled” less frequently, but it also shows a reduction in rate of local water level rise between about 4:45 and about 5:00 a.m.

These gage readings appear to indicate that a levee breach occurred at about 4:45 a.m. near the I-10 Bridge across the IHNC channel, resulting in a local and temporary drawdown of the otherwise rising water levels in the IHNC.

A number of levee breaches occurred during Hurricane Katrina along the north-south channel of the IHNC, so there is no shortage of candidate sites for this breach.

Many of the partial breaches and distressed levee sections of New Orleans East fronting the IHNC (on the east bank of the IHNC) were relatively minor features, with minor flow potential, and could not have accounted for the significant changes observed in the gage readings shown in Figure 8.3. In addition, a number of these features showed evidence of erosion and scour specifically due to overtopping, indicating that water elevations significantly greater than +9 feet (MSL) eventually occurred at their locations.

Similarly, the timing(s) of the occurrences of the two large breaches on the east side of the IHNC at the edge of the Ninth Ward are well-established by eye witnesses as well as by “stopped clock” data, and these two major breaches appear to have occurred considerably later at about 7:45 a.m.

A significant breach occurred on the west side, behind the main Port of New Orleans, due to overtopping and erosion of soil support for an I-wall (see Section 8.2.3.1.). The elevation of the I-wall, and the observed overtopping erosion, indicate that this failure occurred later in the morning as well when the storm surge had risen high enough to pass water over the top of this floodwall (see Section 8.2.3.1.).

That leaves only three candidate breach sites that might have caused the drop in water level rise shown at about 5:00 a.m. in Figure 8.3.

One of these is the breach on the east side of the IHNC at the CSX railroad crossing and roadway crossing over the levee, as described in Chapter 7.

A second candidate site is the pair of breaches that occurred closely adjacent to each other at the south end of the main Port of New Orleans, as described in Section 8.2.3.3. These were large breaches, and might well have had sufficient flow as to account for the drop in water level rise shown in Figure 8.3. In addition, these sections were constructed of highly erodeable lightweight shell-sand fill, and might well have eroded early due to through-passage of seepage flows through the “earthen” levee embankment as the storm surge rose (but prior to full overtopping of the levee embankment at this location.) This is discussed further in Section 8.2.3.3.

A third candidate breach site is the west bank of the IHNC at the CSX railroad crossing, as described in Section 8.2.2. At this location, a steel “storm gate” on rollers had been damaged by a train accident several months prior to Hurricane Katrina, and was away for repair. In lieu of this missing gate, a sandbag levee crest section had been constructed in the opening left by the missing floodgate. The sandbags washed out at some point during Katrina, and this may have been the early breach reflected by the gage readings shown in Figure 8.3. At this same site, flow along the juncture between the railroad embankment and the adjacent embankment fill supporting an asphalt paved roadway passing over the earthen Federal levee resulted in erosion and scour that produced a second breach feature at essentially this same site, as is also described in Section 8.2.2.

In the end, based on the information currently available to this investigation team, any of these three candidate breach sites might have been responsible for the for the observed gage level drops shown in Figure 8.3.

8.2.2 The CSX Railroad Breach

As shown in Figure 8.2, the CSX railroad crosses the IHNC channel immediately to the south of the I-10 Highway bridge. On both the east and west banks of the IHNC, the railroad passes through the levee system by means of a gate through a structural concrete floodwall. Steel gates are used to close these openings during storms.

Figure 8.4 shows the concrete structural floodwall on the west side of the IHNC, at the east edge of the Orleans East Bank (Downtown) protected area. Note that there is no steel gate shown in this photograph. The steel gate at this location had been damaged by a train accident several months prior to Katrina’s arrival, and it was away for repair at the time of the hurricane.

In lieu of this missing steel gate, a temporary sandbag “levee” was erected across the opening. At some point during the storm this sandbag “levee” section either was pushed over by the rising storm surge or was overtopped and washed away.

In addition, erosion occurred at the juncture between the railroad embankment fill and the fill supporting an adjacent roadway passing over the earthen federal levee at this location, as shown in Figure 8.5. This roadway passed over the levee crest to provide access to port facilities on the outboard (water) side of the Federal levee system. This is shown in Figure 8.5, which is a view from the inboard side of this breach showing the erosion of the roadway fill. The elevated I-10 highway bridge is at the left of this photo, and the CSX railroad is just to the left (north) of the roadway. The roadway fill at this location was comprised largely of highly erodeable lightweight “shell sand” fill; a material not suitable for levee fill in a levee protecting a large population (especially without sheetpile cutoff or similar features to prevent erosion.) The flow appears to have passed initially through the pervious gravel ballast supporting the train rails (which is the “low point” at this complicated location), and then undermined the less competent fill beneath the roadway. The resulting flow through the eroded breach then passed to the inboard (protected) side and made its way into the adjacent neighborhood.

The erosion and scour at this conjoined pair of breach locations did not erode the base (lips) of these breach features to a level below mean sea level. Accordingly, although flow passed through this pair of features for a number of hours, the flow eventually ceased as the storm surge (water level rise in the IHNC) eventually subsided.

The failure at this site is an excellent example of a failure produced by multiple adjoining jurisdictions, and a lack of overall coordination of the various system elements constructed and operated by each. The Federal levee system was “penetrated” here by both the railway and the Port roadway, and the interactions of the pervious railway ballast and the highly erodeable roadway fill combined to fail the overall flood protection system at this location. Lack of coordination, and lack of authoritative oversight, of these disparate organizations and their disparate system components was a critical problem here.

It should be further noted that this same site had also failed catastrophically in 1965 during hurricane Betsy, so that the re-failure of this same location represents a daunting case of lack of progress and learning over the intervening 40 years. As discussed in Chapter 7, the east bank CSX rail crossing, which also failed during hurricane Katrina, was also a “repeat” failure (as it, too, had failed during hurricane Betsy in 1965.) The continued failure to recognize and suitably address the hazards associated with these complex “penetrations”, despite their demonstrated history of previous failure, is difficult to understand.

In addition, it is interesting to note that the steel gate was allowed to be removed for repair, rather than requiring it to be fixed in place until a suitable replacement gate (or at least interim replacement gate) could be fabricated and be brought in, so that trains could continue to operate. This created an obvious potential hazard to the safety of the very large community inboard of this rail crossing; placing the safety of many at increased risk. In hindsight; that was a decision that is difficult to justify.

8.2.3 Breaches and Distressed Sections at the Port of New Orleans

Three breaches occurred to the south of the CSX railroad breach on the west side of the IHNC at the main Port of New Orleans. Several additional levee and floodwall sections

were “distressed” or damaged, but did not fully breach along this same section. These breach and distress sites along this reach are jointly indicated as the suite of “Industrial Canal Overwash Sites” in Figure 8.2.

As the storm surge raised the water levels in Lake Borgne, and then pushed the elevated waters (and flow) westward through the “funnel” at the east end of the east/west trending GIWW/MRGO channel between New Orleans East and St. Bernard Parish, large flows and a major rise in water elevations pushed westward along the GIWW/MRGO channel to this channel’s “T” intersection with the IHNC channel, and raised the water levels within the IHNC channel.

This resulted in rising waters rushing directly at the west bank of the IHNC, coupled with overall raising of the water levels throughout the IHNC region. This produced distress, and several breaches, on the west side of the IHNC in the general vicinity of the main Port of New Orleans. The sub-sections that follow will describe each of these in turn.

8.2.3.1 Breach at Rail Yard Behind the Port of New Orleans

The northern-most of these features was a breach in a combined earthen levee and concrete I-wall section, as shown in Figures 8.6 and 8.7. This breach occurred behind the main Port of New Orleans, just to the south of the juncture between the east-west trending GIWW/MRGO channel and the IHNC, so the water pressures and overtopping from the lateral flow from the east-west trending GIWW/MRGO channel were particularly severe at this location, as indicated in hydrodynamic modeling by Team Louisiana (Mashriqui, 2006.)

At the time of our field team’s arrival in late September, this site was already under repair. The field team arrived at this site on the morning of September 30, 2005, and at that time the trench that had been scoured behind the wall on the north end of the breach had been “filled” with clayey backfill and additional backfill had been placed behind the wall to form an additional buttressing berm, as shown in Figure 8.6. A temporary access road had also been placed through the breach, as is also shown in this photo.

Figure 8.7 shows conditions on the north side of this breach, at the same point in time (on September 30.) The interim repair efforts had not yet reached the north side of the breach, and the mechanisms that contributed to this failure were still clearly evident here. As shown in Figure 8.7, significant overtopping had passed over the concrete I-wall and then cascaded down the backside, resulting in erosion of a “trench” at the base of the backside of the I-wall. It should be noted that water falling over an 8 foot high I-wall strikes the ground at a velocity on the order of about 20 to 25 feet per second; sufficient to cause rapid erosion at the point of impact.

The initial height of the compacted embankment fill on the backside of the I-wall prior this erosion can be clearly seen in Figure 8.7 by the soil markings on the I-wall at the left of the photograph. The depth of this erosion (scour) from the elevation of the top of the pre-event I-wall/soil crest contact to the base of the eroded trench was 4.5 feet at the location of the photographer taking the photo of Figure 8.7, and it deepened progressively towards the actual breach location approximately 25 feet to the North. Just before reaching the actual

displaced I-wall section shown in Figure 8.7 this depth of erosion was approximately 5.5 feet, so that the depth of erosion at the location of the actual I-wall failure was likely on the order of 5.5 to 6.5 feet.

The depth of the sheetpiles was unusually shallow at this location, as shown in Figure 8.8. The I-wall “stick-up” had not been large at this location, and it was not felt that very long sheetpiles were needed to support this I-wall by means of cantilever action given its relatively short unsupported length (stick-up). There was no sign of lateral embankment movement at this site, and the sheetpiles and I-wall showed no signs of flexure on their vertical axis (along their length from top to bottom.) The I-wall failed by rigid body “toppling” laterally towards the inboard (protected) side in a “rigid, post-hole” toppling mode as it became progressively unbraced by the erosion of the supporting soil at the inboard toe. Eventually, it became unable to support the water pressures on the outboard (canal) side due to the storm surge and hydrodynamic forces, and the I-wall toppled far enough to permit catastrophic erosion at the main breach section.

As shown in the cross-section of Figure 8.8, the sheetpiles at this section were only 14 feet in length, and were tipped at a base elevation of approximately -6 feet (MSL). As the overtopping water cascaded over the top of the concrete I-wall, the resulting trench eroded to a depth of approximately 6 feet below the original wall/soil crest contact, and the critical section achieved approximately the geometry shown in Figure 8.8. Soil properties are not well established at this location, as site specific investigation was not possible within the budget and time constraints of this independent investigation. Accordingly, soil stratigraphy and soil properties used in our analyses are inferred from the original design data available from the USACE.

Based on the field observation that no major embankment foundation failure was observed, the most significant properties for analysis of this section were the sheetpile sections (which were PZ-22) and the properties of the engineered embankment fill (which was a moderately compacted silty clay of medium plasticity). Shear strength of the embankment fill was modified to determine the strength (and stiffness) at which the observed failure would be expected to occur, and it was found that the I-wall section would be marginally unstable with a fill strength of approximately 600 to 1000 lb/ft². This appeared to be well-representative of the strength of the observed fill, and the failure mode (shown in Figure 8.9) matched well with the field observations. Figure 8.9 shows the results of Finite Element Analyses (FEA) performed using the program PLAXIS; in this case for embankment shear strength of approximately 800 lb/ft², and with a trench to a depth of 6 feet behind the floodwall. This Figure shows calculated displacements, and the rigid toppling mode of wall failure can be clearly seen. Lateral failure of the I-wall results in large part from shearing at the transition between the base of the embankment fill and the underlying foundation soils.

It appears that this failure could have been prevented, simply and at little incremental cost, by installation of concrete “splash pads” or other erosion protection at the base of the inboard side of the I-wall to prevent the observed erosion. Similarly, this failure would have been prevented if the floodwall had been a “T-wall” section, as illustrated in Figure 8.10(b), rather than the less expensive “I-wall” section, as illustrated in Figure 8.10(a). The I-wall sections are supported laterally only by the cantilever action of their supporting sheetpile

walls, and this cantilever action is adversely affected by this erosion. T-walls, on the other hand, have lateral concrete stems at their bases and these are supported both laterally and rotationally by battered piles (providing a much higher level of rotational resistance.)

Instead a significant breach occurred, and floodwaters passed through the adjacent railroad yard and into the adjacent neighborhoods for a number of hours. This breach was located well inboard from the actual edge of the IHNC channel, however, and the breach did not erode its front lip to a depth below mean sea level. Hence, this flow eventually ceased as the storm surge subsequently subsided later in the morning of August 29th.

8.2.3.2 Erosional Distress at Floodgate Structure Behind the Port of New Orleans

Just a few hundred yards to the south of the breach described in the previous section, significant erosional distress occurred at a concrete I-wall and floodgate structure behind the Port of New Orleans. As shown in Figure 8.11, this concrete wall and steel gate structure provided access from the rail yard (on the protected side) to Port facilities (on the water side) which can be closed off by means of a rolling steel floodgate.

Significant erosional distress occurred at each end of this floodgate structure as it “transitioned” to join the earthen levee and floodwall at each end. An example, at the north end of this structure, is shown in Figure 8.12. In this figure, the canal is on the left and the “protected” side is on the right. The trench-like feature at the outboard side toe of the floodwall is the “gap” left when the wall displaced to the right as overtopping eroded a trench on the right side of this wall, laterally unbracing the very short sheetpiles and wall. New fill has been placed on the right side (as an interim repair), so this erosion is no longer visible. The concrete wall of the gate structure has not displaced, as it is supported on a T-wall basis, and the rotational stiffness of the battered piles has been sufficient to prevent wall rotation.

The erosion at the juncture between the concrete gate structure and the adjacent concrete I-wall was locally exacerbated by the disparity in top elevations between these two walls; which acted to locally concentrate the overtopping flow. Erosional distress of this sort, at the “transitions” between differing elements of the flood protection system, was a recurring theme in the damages caused by Hurricane Katrina.

8.2.3.3 Two Adjacent Erosional Embankment Breaches at the North End of the Port of New Orleans

Additional erosional “distress” and two large erosional breaches occurred slightly farther to the south, at the southern end of the main Port of New Orleans.

Figures 8.13 and 8.14 show two views of a large breach through an earthen levee at the contact (“transition”) between the levee and a concrete floodwall section. As shown in Figure 8.14, the embankment fill material is lightweight shell-sand, a material known to be unusually highly erodeable. This type of shell-sand material performed notably poorly at a number of locations during Hurricane Katrina, and is a material not suitable for construction of critical levees protecting large populations. At this location, no provisions (e.g. a sheetpile cut-off, etc.) had been made to prevent catastrophic erosion of this shell-sand fill due to either

overtopping or through-erosion (erosion due to through-flow prior to full overtopping.) In the absence of an eyewitness, it was not possible to discern from evidence at this site whether the embankment was actually overtopped, or whether flow through this highly erodeable fill caused progressive erosional failure prior to full overtopping.

Figure 8.15 shows a second large erosional breach, less than 50 yards from the breach shown in Figures 8.13 and 8.14. This embankment section was also comprised largely of highly erodeable shell-sand fill. A large scour hole can be seen to the right, immediately inboard of this large breach. The massive flows have rippled the asphalt tarmac, and detritus from the eroded shell sand fill is scattered over a large area. As shown in this photo, some of this shell sand detritus has been scooped back into the breach as part of the initial repair.

Although these two adjacent erosional breaches were both of good size, they were both located some distance inboard from the IHNC channel, and neither eroded a pathway all the way back to the IHNC channel that was continuously below sea level. As a result, although both breaches admitted significant volumes of water into the adjacent neighborhoods, flows through these two breaches eventually ceased as the storm surge subsequently subsided.

8.2.4 Summary and Findings

The breaches along the west bank of the IHNC were each “non catastrophic” as none of them eroded or scoured to such depth that their lip dropped below mean sea level. Accordingly, although they admitted significant volumes of floodwaters into the greater Orleans East Bank (downtown) protected area, these flows eventually ceased as the storm surge subsided. Together, these features appear to have contributed approximately 10% to 20% of the overall volume of floodwaters that eventually flowed into the Orleans East Bank (downtown) protected area.

Although they were each “non-catastrophic”, these features each had the potential to cause significant localized flooding and damage. They were also each the result of engineering lapses and/or lapses in oversight during design and construction; none of the failures in this area should have occurred at the storm surge and wind/wave loadings produced at these locations by Hurricane Katrina had proper design and construction features been included.

The removal of the steel floodgate at the CSX Railroad crossing, and the inadequate sandbagging of the resulting “gap” in the overall regional flood protection system should not have been permitted. The steel gate should have been immediately replaced with a suitable and serviceable temporary replacement until the original gate could be repaired and returned. Instead it was missing for approximately three months of the hurricane season. In view of the events during Katrina, it is difficult to justify the decision to remove the gate and thus maintain the “operability” of the railroad line when it placed the “operability” of the flood protection system, and the safety of the community, at risk.

Similarly, the confluence of the CSX railroad embankment and the adjacent roadway both passing over/through the Federal levee system immediately to the south of the I-10

bridge represents one of many “transitions” between disparate flood protection system elements that performed poorly as an apparent result of lack of appropriate oversight and/or poor design with regard to how abutting elements of the system joined at their edges. In addition, highly erodeable shell sand fill was used at this roadway location without suitable cut-off by means of sheetpiles, etc., representing a hazardous condition that should have been caught and remedied prior to Katrina’s arrival.

The erosional “distress” that occurred at the junctures of structural I-wall sections and the structural T-wall gate structure at the rail yard behind the Port of New Orleans represent additional examples of inadequate attention to details at “transitions” between adjacent sections of differing type and geometry.

The two large erosional breaches at the south end of the Port of New Orleans were clearly the result of use of inappropriate fill materials (highly erodeable lightweight shell-sand fill) in earthen embankment sections with no suitable provisions to reduce the obvious risk of catastrophic erosion and breaching. This, too, should have been spotted and remedied prior to Katrina’s arrival. It is not clear whether these two breach sections were overtopped by the rising storm surge, or whether the embankment sections eroded as a result of “through flow” prior to full overtopping as the waters rose within the IHNC.

Finally, the I-wall section breach behind the Port of New Orleans was largely the result of overtopping and subsequent erosion at the base of the inboard toe of the concrete I-wall. This failure could have been prevented, at relatively little incremental cost, by installation of concrete splash pads or other erosion protection at the inboard toe of this floodwall. In addition, there was ample right of way available to construct a somewhat wider (and heavier) levee embankment on the inboard side of this I-wall. The incremental cost of doing so would have relatively small, and that too would likely have prevented this failure.

8.3 The Canal District Failures

8.3.1 Introduction

As the eye of the hurricane began to pass to the northeast of New Orleans, the counterclockwise swirl of the storm winds caused a surge in water levels along the southern end of Lake Pontchartrain. The storm surge along the Pontchartrain lakefront (which peaked at about 9:00 to 9:30 a.m. at an elevation of about +10 feet, MSL) did not produce water levels sufficiently high as to overtop the crests of the concrete floodwalls atop the earthen levees lining the three drainage canals that extend from just north of downtown to Lake Pontchartrain; the 17th Street Canal, the Orleans Canal, and the London Avenue Canal. Three major breaches occurred along these canals, however, and these produced catastrophic flooding of large areas within the Orleans East Bank protected area (as shown in Figure 8.2.)

The first major breach along the drainage canals occurred near the south end of the London Avenue canal, between about 7:00 to 8:00 a.m. The second breach occurred near the north end of the London Avenue canal, and the best current estimates of the timing of this breach are between about 7:30 to 8:30 a.m. The third major breach occurred near the north

end of the 17th Street canal. The main breach here occurred between about 9:00 to 9:15 a.m., but this may have been preceded by earlier visually observable distress at this same location. All three of these breaches rapidly scoured to depths well below mean sea level, so they continued to transmit water into the main Orleans East Bank (downtown) protected area for three days after the initial peak storm surge subsided. More detailed discussions and analyses of these catastrophic drainage canal breaches are presented in the sections that follow.

The resulting flooding of the main Orleans East Bank (Downtown) protected area was catastrophic, and resulted in approximately half of the 1,293 deaths attributed (to date) to the flooding of New Orleans by this event. Contributions to this flooding came from the overtopping and breaches along the IHNC channel at the east side of this protected area, as described previously in Section 8.2, but the majority of the flooding (approximately 80% to 90% of it) came from the three catastrophic failures along the drainage canals at the northern portion of this protected area.

In addition, one of the drainage canals (the Orleans Canal) had not yet been fully “sealed” at its southern end, so that floodwaters flowed freely into New Orleans during the peak of the storm surge through this unfinished drainage canal. A section of levee and floodwall approximately 200 feet in length had been omitted at the southern end of this drainage canal, so that despite the expense of constructing nearly 5 miles of levees and floodwalls lining the rest of this canal, as the floodwaters rose along the southern edge of lake Pontchartrain, the floodwaters did not rise fully within the Orleans Canal; instead they simply flowed freely into downtown New Orleans.

By about 9:30 a.m. all of the levee failures had occurred, and the main Orleans East Bank (downtown) protected area was slowly filling with water. As the northern end filled from the three catastrophic breaches along the drainage canals, water eventually began to pass over low spots in the Metairie Ridge and flowed into the southern zones within this protected area as well.

The sections that follow present more detailed examinations of the performance of the flood protection system in the “Canal District”.

8.3.2 The Lining of the Drainage Canals

There were a number of lapses, errors and poor decisions that led to the catastrophic breaches along the drainage canals and thus the flooding of the main section of metropolitan New Orleans. Several of these began right at the start, in the aftermath of Hurricane Betsy (of 1965) and the flooding caused in New Orleans by that event.

The decision was made, in the wake of Hurricane Betsy, to raise the level of flood protection throughout the region. The three drainage canals (the 17th Street, Orleans and London Avenue canals) were problematic in this regard, however, due to limited right-of-way adjacent to the existing embankments lining these canals.

As described in Chapter 3, the USACE argued (correctly as it turned out) that the low-rise levees lining the canals were not adequately stable as to sustain a significant raising, and

that the preferred solution would be to place storm gates at the north ends of the three canals which could be closed in the event of a Hurricane to prevent storm surge rise within the canals.

This proposal was bitterly contested by the local Water and Sewerage Board, who were concerned that the gates would be under the control of the local Levee Board, and that they might therefore be impeded in their efforts to operate the massive pumps to “unwater” the city from heavy rainfall (which is also a source of frequent, though non-catastrophic, flooding problems in New Orleans.)

The USACE was, in the end, not allowed to install the floodgates, which would have been the technically superior solution, largely as a result of the internecine distrust between the local Levee Board and the local Water and Sewerage Board. In response, the USACE attempted to “exempt” the three canals from the otherwise contiguous levee system around the main metropolitan Orleans East Bank (downtown) protected area.

As discussed in Chapter 3, lobbying by State and local interests next resulted in a Senate rider (inserted clause) on a bill that un-exempted the three canals and specifically required the USACE to raise the level of flood protection along these three canals. This was the first of a number of causative errors that would prove catastrophic here. The canals would remain open to hurricane-induced storm surges at the south end of Lake Pontchartrain; essentially “allowing the enemy (storm surges) right into the backyard” of metropolitan New Orleans.

A second problem now arose. The existing levees were low, and they were relatively narrow as well. Homes had been constructed throughout the area, and the private property at the inboard (protected side) toes of these existing levees left inadequate space for construction of wider levees. Accordingly, a decision was made to raise the level of flood protection by adding reinforced concrete floodwalls to the crests of the existing earthen embankments.

This, in effect, represented a decision to work within the narrow space available rather than purchasing additional property to allow construction of wider, and more stable, levee sections. That was a second issue that contributed significantly to the catastrophic failures that occurred along the drainage canals.

It also resulted in difficulties with regard to both maintenance and inspection, as private homes at the toes of the levees often had property lines interfering with inspection of conditions at the inboard (protected side) toes. In some locations, private property (mainly people’s back yards) extended up the inboard slope faces of the levee embankments, and trees grown on these faces and at the inboard toes of the levees represented an obvious hazard both with regard to seepage erosion and also with regard to the possibility that trees would blow over (in water softened ground) during hurricanes. This would leave large voids (the sizes of their root balls) at a very dangerous location (right at the inboard toes of the levees) at a time when storm surges in the canals were simultaneously rendering seepage erosion, and inboard slope stability, very tenuous. During Hurricane Katrina a number of large trees did indeed topple, leaving dangerous voids at the toes and on the inboard slope faces of the levees along these canals.

In addition, along some sections private homeowners excavated and constructed in-ground swimming pools in close proximity to the inboard toes of these levees, effectively partially undermining them and rendering them less stable. This, too, should have been prevented.

The abutting private properties also led to inspection difficulties, as inspection of conditions immediately inboard of the levee toes is of great importance and private property rights largely prevented inspectors from walking these critical areas. Reports of seepage and wetness at some locations were made to the local Water and Sewerage Board (who were responsible for “unwatering”, and were thus the group to whom such reports were made), but this investigation team has not been able to determine whether these were then passed along to the local Levee Board or to the USACE, to whom they might have represented unanticipated seepage problems warranting further investigation. Certainly the USACE has stated that they were unaware of such reports.

Lack of appropriate control of conditions at the inboard levees toes, and lack of suitable access for inspection and maintenance at the inboard toes, represented additional inadvisable sources of increased hazard.

8.3.3 The E-99 Sheetpile Wall Test Section:

In order to effect the raising of the flood protection levels within the narrow right-of-way available, the decision was made to erect concrete floodwalls at the crests of the existing earthen levee embankments. To facilitate the analysis and design of these challenging sections (on narrowly confined rights-of-way, and on very difficult foundation soil conditions) the New Orleans District of the USACE made an admirable decision to construct a test section and perform a full-scale test of this type of design.

Very similar (difficult, swampy, riverine delta) soil conditions exist nearby in the Atchafalaya river basin (approximately 80 miles to the west), and a site in this area was selected. A sheetpile “I-wall” and contiguous sheetpile curtain, was constructed on the inboard side stability berm of a federal levee in the Atchafalaya basin in a configuration that was very similar to the eventual installation of similar sheetpile-supported concrete floodwalls at the crests of the low-rise levees along the drainage canals in New Orleans (Foott and Ladd, 1977). The swampy foundations soils at this test site were remarkably similar to those at the north end of the 17th Street canal in New Orleans. A sheetpile cofferdam was constructed adjacent to the full-scale test section, and was filled with water to load the test section’s I-wall and its supporting sheetpile curtain.

Two important lessons were learned from this test, and from subsequent analyses (e.g.: Jackson, 1988; Foott and Ladd, 1977; Oner, Dawkins and Mosher, 1997; Oner, Dawkins, Mosher and Hallal, 1997). One was that a gap opened between the sheetpile curtain and the outboard side earthen embankment during loading (by raising of the outboard side water level), and then water penetrated into this gap. This effectively cut the supporting embankment in half, and the water pressures applied against the lower sheetpile sections helped to push the inboard half of the embankment, as well as the I-wall and its supporting sheetpile curtain, towards the inboard (protected) side. This was a failure mechanism that had

not traditionally been considered in the local design of floodwall systems in the New Orleans District. The other lesson was the shape of the failure surface, which was more curved than the deeply plunging three-wedge “planar” failure surfaces considered in the “method of planes” used for analysis of these types of sections in the New Orleans District of the USACE.

Unfortunately, despite publication of these important findings in both internal USACE reports as well as in electronic professional journals (e.g.: Oner, Dawkins and Mosher, 1997; Oner, Dawkins, Mosher and Hallal, 1997), and despite the fact that these studies had been undertaken to facilitate the design of the challenging floodwalls along the drainage canals and the IHNC, neither of these lessons were then incorporated in the subsequent design of the floodwalls along the 17th Street, Orleans and London Avenue drainage canals, nor along the IHNC.

8.3.4 Field Tests for Assessment of Underseepage Risk at the Canals

The USACE also commissioned a pair of local permeability tests at two selected sections along the drainage canals to assess the rate at which changes in water levels within the canals were transmitted through the soils beneath the embankments. The intent here was to assess whether or not it would be necessary to drive the sheetpile curtains deep enough to “cut off” such underseepage flows for the transient loading conditions represented by a storm surge that would raise and then lower the canal water levels within a matter of hours.

The two sections selected were instrumented with piezometers at a series of stations orthogonal to the canals so that the water levels (phreatic surface) could be observed. The canal sections were then excavated to increase the cross-section available for pumping flows. It was assumed that this excavation and deepening would remove the sediment that “sealed” the canals, and would result in an increase in the observed phreatic surface. If little rapid rise was observed, then that would indicate that the increased hydraulic pressures of a transient storm surge would not propagate rapidly under the levees.

There were two critical flaws to this reasoning. One was the assumption that two such tests could suitably characterize the highly variable soil conditions along many miles of the three drainage canals (and also the IHNC). The other was that this testing program failed to note the alternate possibility that the canals were not well “sealed” at all; in which case simply excavating the canals to greater depth would result in no net change in the observed phreatic surface in the piezometers installed inboard at the test sections (the canal water levels would be unchanged by the excavation of the canal bases, and if “steady state” seepage conditions were already established based on full connectivity between the canals and the inboard toe areas then no net change in phreatic surfaces would be observed.)

When the canals were excavated, no significant change in inboard water levels was noted, and it was concluded that underseepage would not pose a significant risk for a short-lived (transient) storm surge. That would prove to be a very serious error, and would result in sheetpiles throughout the system (the three drainage canals and the IHNC as well) routinely being far too short to adequately cut off underseepage. Several major failures along the

drainage canals and the IHNC would result from underseepage during Hurricane Katrina, and the short sheetpiles continue to pose a risk to the remaining sections today.

8.3.5 Water Levels Within the Canals During Hurricane Katrina

Figure 8.16 shows the calculated peak storm surge heights along the southern shore of Lake Pontchartrain based on the most recently available IPET analyses (IPET Report No. 2: April, 2006). These are in close agreement with similar analyses by Team Louisiana along the canal frontage (Kemp and Mashriqui, 2006). As shown in this figure, the storm surge was estimated to be a bit higher at the west end of the “Canal District” than at the east. The water elevations shown in Figure 8.16 are based on the NGVD 29 datum, and must be reduced by about 1 foot to be compatible with the approximate local Mean Sea Level datum used in this report. With this adjustment, the projected peak water levels at the northern ends of the drainage canals are on the order of +10 to +11 feet (MSL) based on these hydrodynamic analyses.

Figure 8.17 shows locations at which relatively reliable high water marks near the mouth of the 17th Street Canal [IPET Report No. 2, 2006]. These high water locations were selected so as to be affected as little as possible by wave action, so that the water levels recorded would be the mean surge height (without wave action.) Based on these data, the IPET study concluded that the maximum storm surge rise at the mouth of the 17th Street Canal was on the order of +11 feet (NAVD 88-2004.66 datum, which is approximately MSL).

Figure 8.18 shows a hydrograph of estimated water elevations vs. time within the 17th Street Canal, based on the hydrodynamic calculations performed by IPET and on observations of water levels at nearby sites [IPET Report No. 2; April, 2006]. This hydrograph peaks at an assumed height of approximately +11 feet, and it peaks fairly sharply between about 9:00 to 10:00 a.m.

Based on the watermark data, our own field observations, and observations and data provided by Team Louisiana (Kemp, Mashriqui and Van Heerden, 2006), our team feel that these are realistic estimates of the surge heights near the mouths of the three key drainage canals (the 17th Street, Orleans, and London Avenue canals), but that they likely slightly overestimate the water levels. Our team has assumed a peak surge height of approximately +10 to +10.5 feet (MSL) at the mouth of the 17th Street Canal, and slightly lesser heights of on the order of +9.5 to +10 feet (MSL) at the mouths of the Orleans and London Avenue canals.

Accordingly, the hydrograph of Figure 8.18, but with a slight reduction of peak surge height (to approximately +10 to +10.5 feet, MSL in the 17th Street Canal, and +9.5 to +10 feet, MSL in the Orleans and London Avenue Canals) will be used for these current studies.

8.3.6 The Orleans Canal

As described previously in Chapter 3, the U.S. Army Corps of Engineers had lobbied and fought for many years to install floodgates to close off the three drainage canals (the 17th Street, Orleans, and London Avenue canals) during hurricanes so that storm surges would not push their way up into these canals. That would have been a superior technical solution, but it was not allowed as there was internecine fighting between the local Levee Board (who are in charge of “protection”; including levees, walls and floodgates) and the local Water and Sewerage Board (who are in charge of “unwatering” by means of pumping for both rainfall and other flooding.) The Water and Sewerage Board were concerned that the floodgates would not be under their control, and so their ability to pump out rainwater from rainstorms (also a cause of flooding in New Orleans) might be obstructed.

As a result of the two disparate local Boards being unable to resolve their differences in the interest of the greater Public good (and safety), the sides of all three drainage canals were instead lined with floodwalls topping the earthen levees along both sides. This, in effect, opened many additional miles of narrow levees and floodwalls atop difficult (and often marshy) foundation soil conditions to storm surges; greatly increasing vulnerability by “allowing the enemy right into the backyard” of this protected area.

An extreme example of the dangers resulting from the poor interaction between the local Water and Sewerage Board and the local Levee Board occurred at the south end of the Orleans Canal.

At the south end of this canal, the main pumping plant crosses the end of the canal as a “T”. Levees and floodwalls provide storm surge protection to an elevation of approximately +13 feet (MSL) along essentially the full length of both sides of the canal, except at the southern end.

The pumping plant is a brick masonry building that was constructed in 1903, and it houses several of the large capacity Woods pumps of that same era. When the water level within the canal rises three to four feet above normal, the operators report that water seeps through the wall of the building that fronts the canal. It is clear that raising the water level significantly higher against the brick face of this old structure would induce water pressures that could collapse this wall.

The obvious solution would have been either: (1) for the Levee Board to construct a floodwall across the south end of the canal, joining to the levees and floodwalls lining the east and west banks of the canal, thus sealing the end of the canal and simultaneously protecting the ancient structure, or (2) for the Water and Sewerage Board to construct a stronger wall, to achieve the same two purposes.

Neither happened.

The Levee Board did not construct the wall to protect the property of the Water and Sewerage Board (and the safety of the Public by closing the base of the canal), and the Water and Sewerage Board did not assist the Levee Board by closing off the end of the canal (and

protecting their own building at the same time.) Instead, an opening of approximately 200 feet in length was left “open” on the east side at the south end of the otherwise continuous levee and floodwall system lining the rest of this canal. A concrete “spillway” section occupies this gap, to prevent erosion from further exacerbating the flows emanating from this hole in an otherwise continuous flood protection system.

Figure 8.19 is a view of the south end of the Orleans canal, showing the brick masonry pumping house, the levees and floodwalls on both sides of the canal, and the “gap” at the south end of the east bank (on the left side of this photo.) Figure 8.20 shows this “gap” from the outboard side, with elevations of key features indicated. Figure 8.21 shows an oblique view from rotation of three-dimensional LIDAR survey measurements (see Appendix A) of this same section. All dimensions, and elevations, are captured by this LIDAR dataset to an accuracy of approximately ± 0.1 feet (or less). The “spillway” section across the open gap has a crest elevation of approximately +6.8 feet (MSL), with a marginally lower “low spot” slightly to the north of the concrete “spillway” section at Elev. +6.5 feet (MSL). The “gap” thus represents a long opening in the otherwise contiguous levees and floodwalls along many miles of both sides of this canal, and with a top elevation of approximately 6 feet below the top of the adjacent floodwalls topping the levees (permitting overflow at approximately Elev. +6.5 to +6.8 feet, MSL.)

As a result, while the storm surge along the southern shore of Lake Pontchartrain was raising the water levels within the full lengths of the adjacent 17th Street and London Avenue drainage canals, the rising storm surge (after reaching an elevation of approximately +6.5 feet, MSL) simply caused floodwaters to flow freely into the heart of New Orleans through this “gap” in the flood protection system.

The opening left at the south end of the Orleans canal resulted in lower water levels toward the south of the canal, but did little to alleviate the storm surge rise at the north end. The lack of failures along the north end of the canal must therefore have been the result of more favorable embankment and floodwall geometries and/or foundation soil properties than occurred along failed sections of the nearby London Avenue and 17th Street drainage canals.

On both sides of this canal there was considerably more right of way available, and the earthen levee embankments along the Orleans canal are considerably wider than those along either the 17th Street or London Avenue canals. Figures 8.22 and 8.23 show views of these levee and floodwall sections along the east and west sides of the Orleans canal. The embankment widths shown in these photos are in strong contrast to the narrower embankments (and crowding from adjacent homes and yards) along the London Avenue and 17th Street canals, as shown for example in Figures 8.109, and 8.24 and 8.30, respectively.

An additional factor working in favor of the stability of the Orleans Canal levees and floodwalls was the fact that the relationship between effective soil overburden stress and resulting soil shear strength in the soft clayey and organic marsh soils near the north end of the Orleans Canal embankments and floodwalls had been better treated during initial analysis and design than it was for 17th Street Canal embankments and floodwalls for similar soils.

8.3.7 The 17th Street Canal

8.3.7.1 *The Breach on the East Bank*

(a) Introduction

One of the most catastrophic failures during Hurricane Katrina was a breach near the north end of the 17th Street Canal, on the east side, just to the south of the Hammond Highway bridge. The location of this breach is shown in Figures 8.1 and 8.2.

Figure 8.24 (which is a repeat of Figure 2.14) shows the use of military helicopters to place oversized bags of gravel into this breach. This photo shows a number of important features at this breach site. In this photo, it can be clearly seen that the inboard side of the levee embankment (on the “protected” side of the floodwall) has translated laterally to the east (to the right in this photo, which is taken looking north.) The translated embankment section is relatively intact along the northern two-thirds of this breach, and appears to have swung much like a door about the northern end. Severe scour and damage to structures on the inboard (“protected”) side at the south end of this feature support this mode; the major rush of inflow was concentrated near the southern end of this breach.

A number of borings and Cone Penetration Test (CPT) probes were performed at this site by the IPET investigation, by Team Louisiana, and by the ILIT investigation team. In addition, several borings had been performed earlier, as part of the initial design studies for the raising of the floodwalls at this location. Figure 8.25 is an approximate plan view of this site, showing the locations of the borings and CPT performed by our (ILIT) investigation. This plan view also shows the locations of a number of important features that help to shed light on the causes and mechanism of this failure. Figure 8.25(a) shows the approximate locations of borings and CPT probes performed at this site by the IPET investigation.

Figure 8.26 shows two views of a cross-section through the heart of this breach along Section A-A' from Figure 8.25. Figure 8.26(a) shows this cross-section before the failure, and Figure 8.26(b) shows this same section after the failure. Nearby cultural features (including buildings, fences, and floodwall sections) as well as boring logs and CPT probes are projected to this cross section for graphical clarity.

As shown in Figures 8.25 and 8.26, the intact levee segment near the center of the breach moved laterally approximately 49 feet. To the inboard side (“protected” side) of the displaced levee embankment sections, three sets of exiting toe overthrust features were mapped, as also shown in these figures.

As shown in Figure 8.26(b), the breach was the result of a translational failure of the inboard section of the embankment, pushed laterally by the water pressures exerted by the storm surge in the canal acting on the outboard face of the floodwall and sheetpile curtain. Figure 8.27 illustrates the sequence of movements associated with this failure, again for the cross-section through Section A- A'. As discussed in the text sections that follow, the rising waters in the canal pushed laterally against the floodwall and eventually (progressively) opened a gap between the floodwall and the outboard section of the levee embankment, as

illustrated in Figure 8.27(b). Water then entered this gap, and increased the lateral push against the sheetpile curtain and floodwall. A shear failure then occurred in the foundation soils beneath the embankment, and the embankment section along with the sheetpile curtain/floodwall slid inboard, pushed laterally by the storm surge as illustrated in Figures 8.27(c) and (d).

Figure 8.28 is an oblique aerial view of this breach section, showing tops of the I-wall sections that “pushed” the inboard section of the earthen embankment (driven by water pressures on their outboard sides), and then toppled backwards towards the canal as the translating levee embankment section finally came to rest and as water pressures equilibrated when the neighborhood filled with water and the storm surge eventually subsided. It also shows two sections of floodwall at the northern end of the failure (the near end in this photo) toppled forward (toward the “protected” side) by the intruding floodwaters at the north end of this breach.

Figure 8.29 shows the tops of the I-wall sections at the very southern end of the breach, which were also left “toppled forward” (towards the inboard, or “protected” side) by the intruding floodwaters passing through the breach opening.

Figure 8.30 shows a collapsed metal shed, with a corrugated roof, that was pushed against the side of the home at 6914 Belaire Drive by “plowing” at the toe of the laterally translating earthen embankment section, as is also shown in Figures 8.26 and 8.27.

Figure 8.31 shows a foundation slab at the toe of the failed section, immediately to the south of the home at 6914 Belaire Drive. The final exiting toe thrust feature rises just at the near end of this slab, which was partially laterally displaced despite being supported by piles, as shown previously in Figures 8.26 and 8.27. Scour caused by the floodwaters also left an erosional depression beneath and behind this slab, resulting in the “pond” shown in the background of Figure 8.31. Also clearly visible in this photo are blocks of peat that were scoured from the foundation strata by the intruding floodwaters.

Figure 8.32 shows a piece of one of the exiting toe thrusts (Toe Thrust #1, from Figure 8.26(b)) at a location between the slab of Figure 8.31 and the home and collapsed shed of Figure 8.30. Figures 8.33 and 8.34 show two views of the other two toe thrust features which occur farther to the inboard (protected) side of the failure (Toe Thrusts #2 and #3, from Figure 8.26(b)).

The general failure mode involved water pushing on the canal side of the floodwall, resulting in the opening of a gap between the sheetpile curtain/floodwall and the outboard side of the earthen embankment. Water then flowed into this gap, and the resulting water pressures pushed the inboard half of the earthen embankment (and the sheetpile curtain/floodwall) sideways. This “cutting the embankment in half, opening a gap, filling it with water, and then pushing the inboard half of the embankment (along with the sheetpile curtain/floodwall)” mode of failure had not been considered or analyzed during the original design of the floodwalls along the drainage canals. It was, however, not an unexpected mode of failure as it had been clearly evinced in the E-99 full-scale test section experiment near

Morgan City in the nearby Atchafalaya basin in 1977 (as described previously in Section 8.3.3.)

In the second IPET interim report (IPET; April 1, 2006) this mode was selected as the likely mode of failure based on stability analyses and centrifuge model testing performed as part of the IPET studies. Our own investigation team had favored this failure mode from the time of the initial post-event field observations in September and October of 2005. It was apparent that this mode had been in operation at this site based on the field observations made at that time. In addition, the same mode had also been in operation, and was “frozen” in place as a partially developed or incipient failure, on the east bank near the north end of the London Avenue drainage canal (see Section 8.3.8), and the field evidence also clearly indicated that this same “half embankment with a water-filled crack pushing laterally” had been the mode of failure at the large breach on the west bank near the north end of the London Avenue Canal (see Section 8.3.8). Also, we had read the E-99 full-scale test section reports, and were aware of the likelihood of this mechanism.

The deeper question is: What was the underlying mechanism that produced the observed failure within the foundation soils beneath the embankment?

Here the findings of our investigation differ significantly from those of the second IPET interim report, and those of the IPET Draft Final Report of June 1, 2006 as well. The IPET report’s finding was that the failure was the result of a largely rotational failure, shearing mainly through the soft gray clays occurring beneath the organic, marshy layers that support the base of the embankment. Our own studies found that there were two mechanisms that were each capable of producing the failure and breach, and that the margins of safety associated with each of these did not differ by large amounts. The actual failure that occurred followed the weakest and least stable of these two mechanisms, and was a largely translational failure along a relative thin but laterally continuous stratum of weak and highly sensitive organic clayey silt silty clay embedded within the “marsh” layer as shown in the cross-sections of Figures 8.26 and 8.27.

An examination of the various soil units, the various potential failure modes, and analyses and explanation of the findings as to the nature of the actual failure mechanism, follow.

(b) Geotechnical Analyses of the Failure

As shown previously in Figures 4.14 through 4.17, the north end of the 17th Street Canal is situated atop largely paludal marsh clays and organic marsh deposits, in an area long riven with erosional drainage features associated with the Lake Pontchartrain basin.

Figures 8.35(a) and (b) present two additional cross-sections showing conditions prior to the failure along Sections B-B and C-C in Figure 8.25. As shown, the foundation soil conditions differed somewhat, but were largely similar along the width of the breach (failure) section.

As shown in Figures 8.26, 8.27 and 8.35, the levee embankment was comprised of two distinct soil fill zones. The upper embankment was a moderately compacted imported brown

clay fill, placed in the early 1970's. This fill had raised the pre-existing levee, which was comprised largely of locally available gray clay fill from the local swamp deposits. Placement of the original layer of gray clay fill dated back to the previous century, and these earlier "historic" fills had consisted of simply piling up locally available gray paludal marsh clays without compaction.

These two embankment fill zones were underlain by a layer of "marsh" deposits. This was actually a relatively complex and layered zone, consisting of strata of peaty organics interbedded with soft, sensitive organic clayey silts and plastic clays with very high water contents, and of varying organic and fibrous organic contents. Cypress tree root systems were common in this mixed "marsh" layer, apparently representing two distinct "stands" or levels of cypress marsh as shown in Figure 8.26(a), and these root systems often interfered with drilling and sampling.

The "marsh" layer was underlain by a transitional layer of progressively less organic soils, with fewer fibrous and peaty inclusions and an increasing fraction of soft, plastic gray clays. Beneath this transitional "intermixing" zone, the foundation consisted of soft, weak gray paludal marsh clays (CH) of high natural water content (natural water contents of $w_o \approx 85$ to 95 %.) These marsh clays were both weak and "sensitive". Sensitivities (the ratio of peak undrained shear strength vs. residual undrained shear strength) were typically on the order of 2 to 6.

These soft gray clays were underlain by fine sands. These sands are adequately strong and competent relative to the softer (and weaker) overlying soil units that they were not involved in the failure. Similarly, although these sands were relatively pervious, this was not a significant issue at this site as they occurred at sufficient depth that they were effectively "capped" by the relatively thick low permeability layer of soft gray clays.

The plan views of Figures 8.25 and 8.25(a) show the locations of borings and CPT probes performed for the original design studies, as part of the IPET investigation, and as part of our own studies. The pre-design and IPET borings generally used 5-inch diameter thin-walled fixed-piston samplers to obtain samples. Most shear strength data reported from both efforts that are currently available to our investigation team are the result of unconsolidated-undrained triaxial tests (UUTX) performed on these samples, although some samples were tested in unconfined compression (q_{unc}). A limited number of in-situ vane shear test results (VST) were also reported for some sites.

Our own field investigations involved primarily the use of 3-inch diameter thin-walled, fixed-piston Shelby tube samples, and laboratory UUTX tests were performed on many of these samples. The Shelby tubes were "modified" prior to use to eliminate the "roll-in" at the cutting end that produces overcutting and then allows lateral expansion of the sample during sample entry into the tubes. It has been shown (e.g. Lunne and Lacasse, 1994) that the use of this type of constant tube diameter, sharp-edged, thin-walled fixed piston sampling with good technique can greatly reduce the disturbance otherwise associated with sampling of the soft, sensitive clayey soils of principal concern at this site.

Some of the borings were sampled continuously and the samples were extruded onsite to examine the stratigraphy and geology in detail. Some of the borings were not sampled at all; instead in-situ vane shear tests were performed at selected depths within these boreholes. Some of the samples were retrieved and brought to the laboratory for testing. Finally, some of the samples were subjected to rather unusual laboratory vane shear testing, and this will be discussed in detail a bit later in this section.

The boring logs for all borings performed as part of these current studies are presented in Appendix B. Laboratory test data, including laboratory vane shear strength test data, are presented in Appendix D. In-situ vane shear strength test data for tests performed within the borings is summarized in Appendix D, and on the boring logs in Appendix B.

In addition to the borings, in-situ and laboratory vane shear and laboratory testing, both the IPET investigation and our own team performed a number of piezocone Cone Penetration Test (CPTU) probes. Logs of the CPTU probes performed as part of our studies are presented in Appendix C.

Figure 8.36 shows a summary of the shear strength test data available to our investigation team for the embankment fill at and near the 17th Street drainage canal breach site. Shear strength of the embankment fill is not of significant direct importance for the conventional overall stability analyses that will follow, as the embankment fill “went for the ride” and was carried along on shear surfaces that sheared through lower, weaker foundation soil units. The strength data from Figure 8.36 was of some importance, however, in selection of properties to model the nonlinear stiffness of these embankment soils in finite element modeling of this levee and floodwall section. The heavy line shown in Figure 8.34 is the shear strength modeled through the embankment fill along the embankment centerline (directly beneath the levee crest) in these studies. The strength lines representing CPT data in Figure 8.36 are based on interpretation of the CPT data using a cone tip factor of $N_k = 12$ in the upper, brown clay fill and $N_k = 12$ in the lower, gray clay fill as well.

Figure 8.37 shows an example of the available CPTU data beneath the central portion of the levee embankment, for “Marsh”, “Intermixing Zone” and “Gray Clay” strata shown in the cross-sections of Figures 8.27 and 8.35. Figure 8.38 shows a similar example of CPTU data, but this time for locations outboard of the toe of the levee embankment. As expected, shear strengths are notably lower here, due to lesser effective vertical stresses resulting from lesser overburden loads.

As shown in Figures 8.37 through 8.40, there are distinct differences between the “gray clay” strata and the “marsh” strata, and these will therefore be treated separately.

Beginning with the deeper unit, the gray clays, it must be observed that our interpretation differs somewhat from that presented in the second IPET interim report. The IPET report assumed that these clays were normally consolidated as they had been protected from desiccation by the overlying swamp deposits. Our interpretation differs, as we found three separate “stands” in the evolution of this layer of soft gray clays and the overlying marsh deposits, with three corollary desiccation-induced overconsolidation profiles associated with these.

The shear strength data based on UUTX tests from the initial design studies, as well as the IPET studies, showed considerable scatter and this was considered likely to reflect the issues associated with sampling disturbance for these soft, sensitive soils. The CPTU data from both the IPET and our own (ILIT) studies, on the other hand, appeared far more consistent within this stratum (as shown in Figures 8.37 and 8.38.) Figure 8.39 shows a typical plot of the pore pressure parameter B_q from a CPTU performed through the crest of the levee ($B_q = \Delta u / (q_t - \sigma_{vo})$), highlighting the value of B_q where the clay appears to be normally consolidated. Figure 8.40 then shows these values of transposed onto the relationships of Lunne et al. (1985) and Karlsrud et al. (1996) to determine appropriate values of the cone tip factor N_{kt} for conversion of CPT tip resistance to undrained shear strength. As shown in this Figure, the value determined for this stratum was approximately $N_{kt} = 12$.

Figure 8.41 then shows the values of $[Su/P]_{OC} / [Su/P]_{NC}$ vs. OCR determined for minerologically similar Mississippi River clays of similar depositional history in Atchafalaya as determined by Foott & Ladd (1977). The SHANSEP exponent for these similar clays was found to be $\lambda = 0.75$, a relatively normal value for clays of this plasticity and character.

Using a value of $N_{kt} = 12$, and $\lambda = 0.75$, the CPTU data within the soft gray clay foundation stratum was then processed to develop plots of S_u/P vs. depth, and OCR vs. depth, for CPT beneath the full height of the levee (Figure 8.42) and inboard of the levee toe where effective overburden stresses were significantly lower (Figure 8.43).

As shown in Figures 8.42 and 8.43, the results show a pleasingly consistent pattern. The clay inboard of the levee toe clearly evinces three “stands” of the marsh development, with three OCR profiles associated with surficial desiccation. The clays beneath the levee embankment loads show just the residual tips of these same three OCR “crusts”, as the clays have been further loaded by the placement of the overlying embankment fill and so are more nearly normally consolidated over most of the stratum. Near the base of this stratum, the clays inboard of the levee toe show a minor degree of overconsolidation associated with secondary compression (as verified by subsequent consolidation analyses using the program PLAXIS which successfully modeled the evolution of this site and accurately reproduced this basal OCR profile).

In establishing the plots shown in Figures 8.42 and 8.43, the value of $(S_u/\sigma'_v)_{NC} = 0.31$ was found to best fit the data. This is a fairly normal value for clays of this plasticity, and it was exactly the same value found by Foott & Ladd (1977) for the minerologically similar clays at Atchafalaya.

The green lines in Figure 8.44 shows the resulting profiles of S_u vs depth within the soft gray clay foundation stratum (a) beneath the crest of the levee, and (b) inboard of the levee toe, based on $S_u/P = 0.31$ and $\lambda = 0.75$. Also plotted on this figure are the CPTU tip resistance data converted to S_u based on $N_{kt} = 12$, and the results of UUTX tests on “undisturbed” ILIT samples, lab vane tests (LVT) on ILIT samples and in situ field vane shear strength tests (FVT). The overall “fit” to all the data is generally very good.

Figure 8.45 then repeats Figure 8.44, but adds the rest of the available IPET and pre-Katrina strength data (including UUTX, FVST and CPT data.) For the “toe” region some

adjustment of this data is necessary in viewing this figure, as some of the IPET data is located such that some portion of the embankment overburden stresses slightly increase the shear strengths for some of the “toe” data; as a result these data (including the CPT) tend to drift to the right (to the stronger side) a bit, especially at depth. Overall, these additional data also well support the relationships developed.

Figure 8.46 then shows the selected value of $(S_u/P)_{NC} = 0.31$ for UUTX, field vane and lab vane tests plotted vs. data for other clays (Ladd, 2003). It also shows the value of $(S_u/P)_{NC}$ for direct simple shear (DSS) tests on the minerologically similar Atchafalaya clays by Foott and Ladd (1973). Both sets of data fit well with the overall background relationship implied for other clays. This suggests that an appropriate scaling factor for the S_u values for conversion from “triaxial” conditions to the DSS stress path conditions that will better represent the stability and deformation analyses for this embankment and floodwall system is approximately 0.80 to 0.84, as shown in Figure 8.46. A value of $S_{u,dss} = 0.82 \times S_{u,tx}$ was used for this soft gray clay in these studies.

As an additional check, the value of $(S_u/P)_{NC} = 0.31$ (for triaxial and in situ vane shear) determined for these clays was also checked against other clays (Figure 8.47.)

Figure 8.48 shows similar treatment of the derivation of the CPT cone factor N_{kt} based on B_q , this time for the “marsh” deposits overlying the soft gray foundation clays. Based on a value of $B_q = 0.25$ to 0.40 , a value of $N_{kt} = 16$ was determined and used to process the CPT data for this unit.

A second approach was also used to also develop profiles of S_u/P vs depth and OCR vs. depth for these marsh deposits, as shown in Figure 8.49. The relationship of Mayne and Mitchell (1978) was used, in conjunction with the available UUTX, LVST and FVST data to iteratively develop relationships for S_u/p vs. depth and OCR vs. depth, as a function of Plasticity Index (PI, %) over the range $PI \approx 55\%$ to 140% , which encompasses the range observed in this complex soil unit. The resulting relationships confirm the classic desiccated OCR crust profile shown previously in Figure 8.41 for this “marsh” deposit.

Figure 8.50 then shows the resulting interpretation, based on all available data, of strength vs. depth within this complex marsh unit for conditions (a) beneath the overburden of the central embankment, and (b) inboard of the toe of the levee. The green lines in this figure represent the final interpreted soil shear strength profiles at these two indicative locations. As with the soft clays, these strengths were, finally, further slightly reduced by multiplying them by a factor of 0.82 to develop the DSS-type strengths needed for the stability analyses performed in these studies.

The red zone near the center of the “marsh” deposits shown in Figure 8.50 is a thin layer of soft, highly sensitive organic silty clay that varies slightly in depth across the profile (and so is thinner than it appears in this figure.) This was the material in which the main lateral translational shear failure occurred at this site.

Figure 8.52 shows a sample of this thin layer at one of three boreholes within the slide region (near to the large, relatively intact displaced levee block) that captured a sheared

sample of this layer. The material is completely remolded and sheared to a fully residual condition with negligible remaining strength, and uni-directional extension and tearing of organic fibers across the sheared zone clearly indicate the shear failure within this sample.

This layer is typically only one to several inches in thickness, but was found to be laterally continuous across essentially the full site (as well as at the distressed section on the opposite, west side of the canal.) It is exceedingly difficult to spot, and to sample, because it is closely overlain (and even partially mixed with) a layer of leaves and twigs and bark that is typically also one to several inches in thickness, as illustrated on the auger stem in Figure 8.51. The very dark, shiny material also coating the auger stem in this photo is the sensitive organic silty clay and indicates that we have just drilled through the layer in question (and so now have to move our hole laterally a few feet and re-drill to attempt to sample it.)

This layer of sensitive organic silty clay is the result of a previous major storm that churned up organics and sediments, mixed them with the locally prevalent clays, and also greatly (temporarily) increased the salinity of the water so that the ensuing deposit is unusually heavily flocculated. The result is a material of low strength and extremely high sensitivity (sensitivities of between about 10 and 20+.)

The same storm was accompanied by winds that knocked down leaves and twigs and bark (and other organic detritus), accounting for the closely overlying layer of organic impediments that “mask” this thin layer.

Figure 8.53 shows a plan view of the site, highlighting with red the 10 locations at which this layer was positively identified. It was not always possible to positively identify this thin layer in CPT, as the strength of this layer is not much less than that of the closely overlying and underlying soils; it is the combination of low strength and high sensitivity that made this thin layer so dangerous. “Thin layer” effects also made spotting this layer in CPT (based on tip resistance) difficult. The best initial “marker” or signature of the presence of this layer was found to be a positive spike in friction ratio; as the sleeve continued to drag through the overlying and underlying deposits but the tip resistance dipped a bit.

Figure 8.54 shows a photo of an “undisturbed” sample of this sensitive organic silty clay. The local clays have a gray, peanut butter-like appearance and consistency. They are not highly shiny, but rather semi-glossy, and their stiffness and texture are not unlike peanut butter. The sensitive organic clay, on the other hand, is dark and has a very shiny and translucent appearance; much like “jelly”, as shown in Figure 8.54. In Figure 8.54, hints of the organic detritus that closely overlies and masks access to this thin layer can also be seen.

Two approaches were taken to attempt to characterize the strength (and stress-deformation) behavior of this material. At any location, the precise depth of this layer was first determined by drilling to encounter it. One approach was then to move the drill rig laterally several feet and to re-drill to within approximately one foot of this layer. A 3-foot long Shelby tube, 3-inches in diameter (and modified to eliminate the turn-in that produces overcutting at the mouth) was then used, with a fixed piston system, to drive the tube approximately two feet past the target layer so that more competent underlying soils would “plug” the bottom of the tube and permit careful withdrawal of a sample. Otherwise, the

samples remoulded upon attempted withdrawal and slopped out of the base of the tube making sample recovery nearly impossible.

The samples thus obtained were then taken to the lab at the University of California at Berkeley, where they were subjected to an unusual process, as illustrated in Figure 8.55. The tubes were cut off in 2-inch increments, and a small spoon was used to carefully dig ahead into the remaining tube. When the tell-tale organic detritus was encountered digging stopped and the organic material was hand-plucked from the tube to daylight the underlying sensitive layer. A lab vane shear test was then performed.

The second method used to evaluate the strength of this material also began by pre-locating the precise depth of this layer, usually by sacrificially “oversampling” it (to plug the base of the tube to foment retrieval) and then extruding the sample to determine the precise location of the layer. A second, adjacent hole was then carefully hand augered, and an in situ vane shear test was performed using a shallow-bladed vane. Insertion disturbance, and obstruction by unremoved organic detritus (mixed in the top of the layer) sometimes defeated this effort, often making multiple attempts necessary. Unacceptable insertion disturbance was apparent when the characteristically brittle peak to residual transition was absent and the material exhibited only residual strength.

Figure 8.56 shows typical stress-displacement plots for tests on the thin layer of highly sensitive organic silty clay, and on the local deposits of sensitive gray clay. As shown in Figure 8.56(b), which shows normalized behavior in the form of shear strength divided by maximum shear strength on the vertical axis, the sensitive organic clay was more highly brittle, failed at lower displacement, and exhibited even more pronounced sensitivity and rapid post-peak strength degradation. It was the combination of low strength, and this very brittle sensitivity, that caused this material to “capture” the failure surface at this site.

Finite element analyses were performed for this levee and floodwall section using the program PLAXIS. Figure 8.57 shows the principal parameters and the mesh used for these analyses. The gray foundation clays (CH) and the “marsh layer” were modeled using the “soft soil” effective stress model within PLAXIS, and the soil parameters used were fitted to the values of S_u/p vs. OCR as described previously to match the evaluated strengths of these units and their distribution.

It was necessary to establish the stress state at the end of incremental construction and consolidation of the embankment and foundation. Initial overconsolidation profiles due to desiccation and secondary compression were input, and embankment construction was modeled in two stages (the “historic” fill, and the more recent engineered top fill), and both the OCR vs. depth and the settlement pattern (the bowl shaped pattern at the base of the oldest fill) were well matched to the observed field conditions. Figure 8.58 shows the settlements calculated at the end of initial construction and consolidation.

The front lip of the embankment was then “excavated” and the floodwall installed (as with the actual field case), and displacements were re-zeroed to prepare for the remaining analyses to follow.

Water levels within the canal were incrementally raised, and within a range of stiffnesses considered reasonable it was found that initiation of “gapping” between the outboard toe of the floodwall and the outboard embankment section typically initiated at a surge elevation of between about 7.5 to 8 feet, as illustrated in Figure 8.59. This Figure shows normalized shear strain contours, with the red color indicating shear strains equal to or greater than the shear strain to “peak” shear strength (and thus localized failure.) As shown in this figure, with a water elevation of +8 feet (MSL) gapping has opened partially down the front face of the sheetpile curtain (on the outboard, or water side), and the thin, sensitive organic silty clay layer has already sheared to failure along a short segment inboard of the crest of the levee.

If one looks very carefully at Figure 8.59, a second “lighter” area can be seen beneath this shear zone, representing the beginning of shear deformations along a more “rotational” shear surface passing through the deeper soft gray foundation clays (CH). A dashed line has been added to indicate this surface. This deeper, and more rotational failure surface has a calculated factor of Safety only slightly higher than that calculated for the upper sensitive organic silty clay layer, and this deeper surface represents the failure mechanism favored by the IPET studies reported to date.

As the analysis began to calculate the progressive development of tensile effective stresses between the front of the sheetpiles and the soil, the mesh was revised to model the development of a “gap” between these, and the intrusion of water into the gap as well. Once this “gapping” began, it then developed rapidly. Figure 8.60 shows the situation with an additional foot of storm surge rise to Elev. + 9 feet (MSL) based on our best estimates of the soil parameters. As shown in this figure, the gap has now extended nearly to the base of the sheetpiles. Further extension of the gap is temporarily held up by the malleability of the marsh soils, but further gapping does not provide significant additional lateral water pressures against the front of the sheetpile curtain because the lateral permeability of the “marsh” deposits is relatively high. At this stage, the shear failure along the thin layer of sensitive organic silty clay is well developed, and embankment movements are now significant. This figure also shows quite clearly the deeper, more rotational failure surface that represents the second least stable mechanism at this site (the mechanism favored to date by the IPET studies.)

Figure 8.61 shows calculated displacements for a surge height of 8.5 feet, with displacements exaggerated times two for clarity. Initially, the floodwall tilts slightly forward as it compresses the soils a bit. As sliding then develops, the floodwall base begins to move along with the displacing embankment and the whole moving mass (inboard embankment section, floodwall and sheetpile curtain) displace laterally together, as shown previously in Figure 8.27.

Figure 8.62 shows the Factors of Safety calculated (by $c-\phi$ reduction) using PLAXIS for a variety of water levels in the canal. Three cases are presented: (1) failure dominated by the thin layer of sensitive organic silty clay, but without gapping between the sheetpile curtain and the outboard side soils, (2) a more rotational failure through the deeper soft gray foundation clays, again without gapping, and (3) failure dominated largely by the upper

sensitive organic silty clay layer, but this time with a water-filled gap on the outboard side of the sheetpile curtain.

As shown in this figure, the Factors of Safety for the upper lateral shear failure, and the deeper more rotational failure, are not very different. The heavy red line shows the best-estimated path to failure at this site. Based on these analyses, it appears that gapping would have developed at a surge height of between about 7.5 to 9 feet (MSL), and the intrusion of water into this gap would have increased the lateral forces and rapidly driven the section to instability.

Figure 8.63 shows the cross-section and principal soil properties used to perform more classical limit equilibrium analyses (using Spencer's Method, cross-checked against Morgenstern's and Janbu's Methods) using the program SLOPE/W.

Figure 8.65 shows the most critical failure mode for the "no gapping" case with a surge height to Elevation +6 feet (MSL). The PLAXIS analyses had shown very little likelihood of gapping at this water elevation, and this probably represents the best estimate of Factor of safety for this surge height. As shown, the calculated Factor of Safety is $FS = 1.51$ for this case, and as shown in Table 8.1, the associated probability of failure for this surge height is approximately $P_f = 0.01$. These calculated low probabilities of gapping and of failure are reassuring, as the water in the canal had previously reached an elevation of approximately +6 to +6.5 feet (MSL) during previous storm surges, and no gapping or failure had occurred in those events.

Figures 8.66 and 8.67 show the most critical failure surfaces for a surge to Elev. +9.5 feet (MSL) for (a) a shallow translational failure dominated by sensitive organic silty clay layer, and (b) a deeper, more rotational failure through the soft gray foundation clays. In both analyses, a water-filled gap was modeled at the outboard side of the sheetpile curtain. This water elevation is approximately the maximum elevation achieved (maximum surge at this location is estimated by our team to be approximately Elev. +9.5 to +10 feet, MSL). The calculated Factors of safety are again similar for both modes, and the shallow lateral translation along the sensitive organic silty clay again provides the lower Factor of Safety.

Figure 8.68 shows calculated Factors of Safety for various water elevations (Spencer's Method) for the four cases of principal interest: (a, b) lateral translation along the sensitive organic silty clay layer, with and without a water-filled gap, and (c, d) deeper and more rotational failure, again with and without a water-filled gap. The solid red line again shows the best-estimated path to failure at this site, this time based on the suite of limit equilibrium analyses.

It is challenging to make an estimate of the probability of failure at any given canal water level, as there are numerous uncertainties involved, and some of these are cross-correlated. The principal uncertainties are those associated with shear strengths of the foundation soils, and also with the "representative" shear strength that can be mobilized at any given moment by the very sharply strain-softening soils (especially the highly sensitive, thin organic silty clay layer.) Additional significant uncertainties are those associated with the likelihood (and severity) of opening of the water-filled gap at the outboard side of the

floodwall and its supporting sheetpile curtain, and the unit weights of some of the soils. These were not all accurately reflected in these probabilistic estimates, and as a result the overall uncertainty is expected to have been somewhat underestimated.

One important set of variables are the shear strengths of the various soil units controlling each of the potential instability modes. Each of the conventional limit equilibrium analyses performed was performed using probabilistic variation of these shear strengths. The coefficient of variability in soil shear strengths was taken as log-normally distributed, and was estimated as approximately $COV \approx 30\%$ for the soft gray clays, and $COV \approx 40\%$ for the thin, sensitive organic silty clay layer. The resulting distributions of probable factor of safety are shown (approximately) graphically in Figure 8.68(a) for both the “un-gapped” case and the case of a water-filled gap at the outboard side of the sheetpile curtain. These are only approximate, as they do not precisely fit themselves to any single well-known distribution.

The next critical uncertainty is the probability of cracking. The probability of cracking cannot be calculated or evaluated in any closed-form manner, and so requires a judgmental estimate based on the preceding finite element analyses (and supported in part by the observed field behavior). It should be noted that the stiffnesses used in the PLAXIS analyses to estimate inception of cracking are a bit time dependent, so that a slower rising and falling storm surge would be a bit more deleterious here. The inception of cracking was not taken as the point at which cracking “occurred”; instead cracking was taken to be significant when the crack propagated more than halfway towards the base of the sheetpile curtain. Figure 8.68(b) shows the judgmentally derived estimates of probability of significant crack formation as a function of rising canal water elevation. The upper and lower bounds shown were inferred to represent approximately $\pm 3\epsilon$ values.

Monte Carlo simulation was used to estimate the distribution of the factor of safety considering the analysis with and without a gap and the probability of a gap forming. The formulation is as follows:

$$P(FS) = P(FS_{NG}|NG)P(NG) + P(FS_G|G)P(G)$$

This equation reads; the distribution of the factor of safety is equal to the conditional distribution of the factor of safety for the levee with no gap multiplied by the probability of there being no gap, plus the conditional distribution of the factor of safety for the levee with gap multiplied by the probability of there being a gap. The conditional distribution of the factor of safety with or without a gap is based on the mean and standard deviation from stability calculations. For the probability of gapping, which can also be considered a transition function from no gap to gap conditions, a mean probability function and upper and lower bounds were estimated from Figure 8.68(b). The above equation is for any single canal water elevation.

All the distributions were treated as Gaussian based on observation of the data. The gap and no gap conditions were considered statistically independent scenarios. A Monte Carlo simulation was run for 10,000 samples. Typical simulation results are shown in Figure 8.68(c) for a single depth increment. The first plot is a histogram of the simulation results of the factor of safety for no gap conditions $[FS_{NG}|NG]$, the second is for gap conditions

[$FS_G|G$], the third is the probability distribution of a gap (no gap) occurring [$P(G)$ and $P(NG)=(1-P(G))$], and the fourth is the total distribution of the factor of safety [$P(FS)$].

Figure 8.68(a) showed the distributions of factor of safety for the gap and no-gap cases (separately) as a function of rising canal water levels. Figure 8.68(d) repeats this figure as a background, but adds the now calculated distributions of conjugate overall factor of safety as a function of rising canal water levels, showing how the conjugate distribution “transitions” from the un-gapped to the water-filled gap case. Based on these approximate simulations, the resulting probabilities of failure at any given canal water elevation are then as shown in Table 8.1.

As shown in Table 8.1, the probability of failure was found to be very low for surge heights of less than about Elev. + 7 feet (MSL), and they rise rather quickly as the surge elevation passes above about + 8.5 feet (MSL). Failure at the estimated actual maximum surge elevation of approximately + 9.5 to + 10 feet (MSL) is calculated to have had a likelihood, on the order of $P_f \approx 0.8$ to 0.9. Failure at the originally intended “design” surge height of Elev. + 12.5 feet (MSL) was essentially certain.

Finally, Figure 8.69 shows a comparison between the observed failure mode and the rotational mode determined by IPET. There have been a number of IPET representations (to date) of their failure mode, each varying slightly as to actual depth and dimensions, but all were semi-rotational failures through the soft gray clay stratum underlying the marsh deposits. The rotational surface shown in Figure 8.69 is a somewhat “average” representation of these various failure surfaces, from both the second interim report (IPET, April 1, 2006) and the more recent Draft Final Report (IPET, June 1, 2006.) The rotational IPET failure is superimposed, as carefully as possible, onto our own investigation’s more detailed cross-section. The two modes are not wholly dissimilar, and both lead to low factors of Safety.

More detailed examination of the IPET mode, however, shows it to be problematic with regard to agreement with key field evidence. The rotational IPET mode would have left the chain link fence at the edge of the crest road (on the displaced intact levee block) rotated backwards, but as shown clearly in Figure 8.26 (and Figure 8.69(a)) this crest fence was essentially perfectly vertical at the end of the displacements. Massive rotation would have been necessary to produce the observed very large lateral displacement of the upper “intact crest” section of the levee (lateral displacement of up to 50 feet), and this would not have been feasible with the IPET failure mode. Also, the IPET rotational mode would have significantly back-rotated the floodwall; but the floodwall instead traveled the full lateral distance (50 feet) in contact with the displacing levee embankment section, and then toppled backwards as the water pressures began to equilibrate (as illustrated in the top of Figure 8.69, and in Figures 8.28 and 8.69(a)). The IPET mode also fails to explain the large lateral extent of the mapped toe exit features, and the multiple toe thrust features, as shown in Figures 8.26, 8.27, and 8.32 through 8.34.

Most importantly, the shear failure along the thin, highly sensitive organic silty clay stratum was confirmed at several locations based on remoulding and also uni-directional extension and tearing of organic fibers; conclusive evidence as to the occurrence of massive uni-directional shear failure along this stratum within the marsh sequence.

The failure of the IPET investigation to discover the critical thin stratum of sensitive organic silty clay that was the principal culprit in the failure at this site represents an important lesson both for current geotechnical practice at large, and also for subsequent design studies for the levees and floodwalls of the new Orleans regional flood protection system. The IPET studies drove numerous geotechnical borings and CPT probes right through this stratum (see Figure 8.25(a)) but did not discover it. That was, in large part, because the crews performing the field borings and CPT were “separated” from those who had performed the initial IPET post-event forensic field studies, and both sub-teams were separated from the expert “engineering geology” team also working on the IPET studies. The analysis sub-team was a fourth, separate group. There were geological experts on the IPET team who could certainly have pointed out the possibility, and even likelihood of such a layer if they had been asked. Instead the four sub-teams performed their tasks largely separately, without adequate interchange of knowledge and findings.

Our own investigation team took a wholly different approach. We began by carefully assessing the visually observable surface forensic evidence at this site in the wake of the failure, and by back-tracking through the original (pre-Katrina) field and lab data for this site. We also studied the challenging geology of the region (including seminal publications by the USACE’s geological experts.) Based on all of this, our site team (which included senior investigative team members right out on the drill rigs) went in search of an unusual layer, of high sensitivity, and considerable lateral extent, that would have been capable of producing a lateral translational stability failure with toe thrust features extending to unusually great distances inboard of the original levee toe. The suspected depth of this stratum, inboard of the levee toe, was fairly shallow; probably between several feet to as much as 10 feet at most. We encountered and identified the critical layer with our first boring, and then sampled and tracked it across the site in a total of 11 borings and CPT’s.

Two important lessons here are: (1) the importance of fully integrating all phases of field investigation, laboratory testing, analysis and design (and the team members performing these), and (2) the importance of suitably involving expert engineering geologists in all phases of site investigation and site characterization, as well as the other project phases. These two things are, unfortunately, not always done in contemporary geotechnical practice, and they are also not the norm for many contemporary Corps design studies which tend to be relatively segmented (as were the IPET studies in this case.)

Overall, it can be concluded that there were two potentially critical failure modes at this site, but that the lateral translational failure along the sensitive organic silty clay layer within the “marsh” deposits was the weaker of the two, and that this was the mode of failure that actually occurred at this site.

(c) Initial Section Design Studies

The obvious next question to address is then how the original design studies failed to note this. The answer is a bit complex as a number of poor judgements and errors contributed to the mis-perception of the original “design” section as being adequately stable (and reliable)

for targeted design canal water elevations significantly higher than those that caused the actual failure (the design canal water level was Elev. +12 feet, MSL). The original design studies have been reviewed, and the following are significant errors and poor judgements during initial design that contributed to this failure:

1. Figure 8.70 shows the longitudinal cross section along the segment of the east bank of the 17th Street Canal as developed for the original design studies. An early error in the design process was the use of borings that were too widely spaced to attempt to characterize challenging and complex foundation geology. The savings achieved by not performing more borings now appear miniscule relative to the cost of the catastrophe that has ensued. [It should also be noted, however, that even the borings that were performed appear to have been sufficient as to correctly predict the failure, if the resulting data had been suitably processed and then used in the ensuing analyses.]
2. The longitudinal section of Figure 8.70 was prepared by the USACE, and was based on a number of assumptions; including the assumption the “marsh” deposits were typically flat-bottomed. The history of previous drainage channel erosion across this area would lead to the expectation of likely non-level transitions even for swamp bottoms, and Figure 8.71 shows our own team’s re-interpretation of the original (sparse) longitudinal data to develop an alternative longitudinal subsurface soil profile. This difference in interpretation might be considered the second problem at this site during original design.
3. The USACE then passed the design on to outsourced engineers, who developed the strength data and interpretations for analysis of stability of the intended levee and floodwall section. A major problem occurred here, as data from far too large a lateral distance was eventually transposed to the design analysis cross-section. In the vicinity of the actual failure, there are only 5 sample locations shown within the critical “marsh deposits” (in the 4 borings shown intersecting this unit.)
4. Two of the sample locations shown within the “marsh” deposit of Figure 8.69 were non-recovered samples, and at approximately the same depth in nearly adjacent borings. This is the location of the sensitive organic silty clay layer that actually caused this failure and breach. Failure to note the importance of the non-recovery of testable samples, and in two nearly adjacent borings at essentially the same elevation, should have represented a red flag and an effort should have been made to further investigate this location.
5. Figure 8.72 shows the stability calculations for the critical section nearest to the actual breach and failure. The limit equilibrium method used for these was the “Method of Planes”, a three-wedge analysis with conservative side force assumptions. This method continues to be preferred by the New Orleans District of the USACE, but it is now a relatively archaic anachronism given the availability of more accurate methods and the availability of the simple computer programs necessary to run these. The method itself provides a slightly conservative answer so long as the most critical failure surface can be closely represented by the steeply plunging wedges at the front and back, and by the horizontal surface in between. In the original design analyses, layers were assumed to be laterally horizontal, so

this analysis was a good fit for the cross-sections analyzed. Unfortunately, the actual stratigraphy was not horizontally layered (see for example any of the cross-sections analyzed in these current studies), so this method was poorly suited to the finding of the failure mechanism that was actually most critical.

6. And the assumption of laterally horizontal layering was itself a major problem too. It was born of necessity, as no borings had been performed significantly off the embankment centerline alignment to permit development of full lateral cross-sections. Again, the minimal savings on exploration and testing costs here pale relative to the costs of the catastrophe that ensued. Stratigraphy is a vitally important issue, especially given the low strengths of many of the foundation soils. Looking at the cross-sections at the 17th Street canal breach site as analyzed in this current study, for example, one will note a subtle “bowl shaped” settlement pattern at the base of the embankment fill, and a corresponding bowl shape to the critical sensitive organic silty clay layer just beneath it. Without this “bowl shape”, the original embankment would have been unstable during initial construction; it would have slid sideways on the sensitive layer if that layer had been horizontal. Instead the layer dipped in the center, so that the evolving embankment would have had to slide up a small slope (up a hill) to fail during construction. Minor changes in stratigraphy details can have a major impact on overall stability on these soft, weak soils. Use of “assumed” horizontal layers therefore missed a vitally important element of the problem.
7. Figure 8.73 shows the now well-circulated summary of strength data for stability analyses at this section. The data are based on UU triaxial tests and on vane shear tests. Scatter in the data is considerable, and is likely due in large part to sampling disturbance issues for these sensitive soils. Most samples were obtained from borings through the crests of the levees (the most accessible location) and so represent strength information for locations under full embankment overburden stresses. The solid lines in this figure show the strength interpretation used in the actual design analyses. This line represents an unconservative assessment of the data points presented, in both sides of the figure, even without allowance for the additional effects of overburden stress reduction away from the levee centerline. This interpretation is especially unconservative at elevations of between + 10 feet to – 10 feet (Cairo datum) in the figure at the right, and between -10 feet to -30 feet (NGVD datum) in the figure at the left. These both represent the same 20 foot range of critical elevations, which correspond approximately to Elev. -10 feet to -30 feet (MSL), and this is the region in which strengths are important in the “Method of Planes” analysis performed for this location in the original design studies. As shown in Figure 8.74, a majority of the available shear strength data is lower than the shear strength actually used for the stability analyses in this critical depth range; violating customary “Corps” procedures in this regard. (Corps procedures generally require that approximately 1/3 of the data fall below the strength used for analysis and design, and that 2/3 of the data be greater.) As shown in Figure 8.70, the resulting calculated Factor of Safety was found to be $FS = 1.30.....$, barely enough to satisfy the design criteria which required a FS of at least 1.3 for the case of “transient” storm surge loading. It is very difficult to

justify the apparently unconservative strengths selected in this critical elevation range based on the data presented.

8. Figure 8.74 is a repeat of Figure 8.73, but with additional red and blue lines added to illustrate another major error made in determination of shear strengths for stability analyses. Shear strengths of soils are very strongly a function of effective overburden stress, so the samples obtained from beneath the overburden of the embankments would consistently overestimate the strengths under the levee toes, and in the “free field” out beyond the levee toes. This fundamental principle of soil mechanics was well-known in local practice in the New Orleans region at the time that these analyses were performed. However, it was ignored in the original design studies at this section, and the result was a massive additional increase in the unconservative error in the overall stability analyses. The blue lines on Figure 8.74 represent our own team’s assessment (as described in preceding sections) of the shear strength vs. depth beneath the crest of the levee, and the red lines represent our assessment of the shear strength vs. depth inboard of the levee toe. The contrast is very significant, and the unconservatism involved in the mis-use of strengths from “beneath the full levee overburden” to model conditions beneath and inboard of the levee toe is readily apparent.
9. Despite having adroitly invested significant funds and effort in the E-99 test section (near Atchafalaya; see Section 8.3.3) to perform a very well-designed full-scale field test on appropriate foundation soil conditions, the results of this field test of a model floodwall/sheetpile curtain in a levee embankment founded on weak marshy soils were not subsequently used (as had been intended.) The failure mechanism disclosed by this field test was the opening of a gap at the outboard side of the sheetpile curtain, the filling of this gap with water, and thus the resulting exertion of increased lateral water pressures against the sheetpile curtain. This mechanism, which proved to be the actual field failure mechanism at this site, was not among the suite of cases/mechanisms analyzed in the original design studies.
10. And the use of a design Factor of Safety of only 1.3 was also a major problem. As discussed in detail in Chapters 11 and 12, this was far too low a value for a system protecting a large urban population. This value has a history of development that is traced in Chapter 12 back to use for design of levees protecting agricultural lands in the first half of the last century, and failure to update this in the face of both the passage of time and the increased level of potential consequences associated with flood protection of a major urban area was a significant lapse that left little room for the other errors and poor judgements cited above.

Calculations using the data available at the time of the initial design, and using analysis methods widely available and in common use at that time (though not necessarily within the new Orleans District of the USACE), clearly indicate that this section would be expected to be unstable at canal water levels less than those for which the design was intended (water level of less than Elev. +12 feet, MSL). The more sophisticated analyses employed in these current (ILIT) studies give more precise answers, but this level of sophistication was not necessary to demonstrate overall deficiency of the original design.

8.3.7.2 *Distressed Section on the West Bank*

There is a “distressed” levee and floodwall section on the west bank of the 17th Street Canal, across from the large breach discussed above. This “distress” was visually minor, but this section was studied both as a check of the ramifications of “minor” visually observable distress, and also because it provided an opportunity to see if the same analysis methods that correctly predicted the failure on the east bank could also accurately predict the observed performance of a second section that it was hoped would be somewhat similar.

Figure 8.75 shows measurement of observed lateral wall offset at the point of maximum offset. Wall tilt is less than 0.75 inches, and the maximum lateral offset is approximately 3.5 inches.

As shown previously in Figure 8.25, only a few borings and CPT were performed at this distressed section on the west bank of the canal, so data is sparse. Figure 8.77 presents the interpreted cross-section used for analysis at this site. The same basic sequence of strata observed on the east bank are again present, but the details of the stratigraphy differ a bit.

Passing quickly through intermediate details (as were presented in detail in Section 8.3.6), the same procedures were used to process and interpret the limited available data, and this was supplemented by the knowledge gained from across the canal. Figures 8.78 and 8.79 show an example determination of the value of $N_{kt} = 12$ for the soft gray foundation clay (CH), and this matches with this same deposit on the east bank. Using the same methods, and the same SHANSEP exponent $\lambda = 0.75$, Figure 8.80 shows the iterative processing of the CPTU data to develop profiles of S_u/p vs depth, and OCR vs depth for this clay unit. These too match well with the east bank deposit data.

Figure 8.81 then presents our SHANSEP-based profiles of strength vs. depth (a) beneath the crest, and (b) at the toe, along with the available strength and CPT data. The fit with the available data is excellent.

Figure 8.82 shows the use of the correlation proposed by Mayne and Mitchell to develop profiles of S_u/P vs depth and OCR vs. depth within the “marsh” deposits overlying the soft gray clays. This matches well with the CPTU-based interpreted OCR profile within this stratum, and with the data from the east bank as well.

Figure 8.83 presents the resulting overall profiles of strength vs. depth within the marsh deposits (a) beneath the crest, and (b) at the toe, along with all available data (including CPT tip resistances interpreted using $N_{kt} = 16$). The thin layer of sensitive organic silty clay was encountered in one boring, again at the approximate mid-point in the “marsh deposits, and again closely overlain by leaves and twigs. This sample is shown in Figure 8.76. Strengths for this thin layer were based on S_u/P values from the east bank deposit. This thin layer was not critical at this west bank site, as the sheetpiles penetrated well below this sensitive layer and so forced a deeper, more rotational failure through the soft gray clays to be the most critical mode.

Once again, all shear strengths determined represented triaxial or vane shear strengths, and these were reduced slightly (multiplied by a factor of 0.84) to develop shear strengths suitable for the direct simple shear (DSS) dominated shear surfaces to be evaluated.

Figures 8.84 and 8.85 show the most critical failure surfaces (without gapping) for a storm surge level of +9 feet (MSL) for failure (a) to the top of the soft gray clay, and (b) within the lower marsh deposits. These both give low Factors of Safety, but the failure through the lower marsh strata is the more critical case.

Figures 8.84 and 8.85 show these same two potential failure modes, again for a storm surge elevation of +9 feet (MSL), but this time with an assumed water-filled gap at the outboard face of the sheetpile curtain. Once again the lower marsh units present the more critical mechanism.

Figure 8.88 shows calculated Factors of Safety vs. canal water elevation for the failure through the lower marsh stratum, both with and without gapping. The heavy red line in this figure shows the best estimate of the likely critical failure path, based on these limit equilibrium analyses. It is judged that gapping is most likely to be initiated at surge elevations of approximately +10 to +11 feet (MSL) as the Factor of Safety (without gapping) drops below about 1.25 to 1.35. Gapping was relatively unlikely during Katrina (max surge level ~ +10 feet, MSL), and indeed no gap could be seen.

Based on these analyses, probabilities of failure were again estimated using the same procedure as described previously in Section 8.3.7.1. Figure 8.88(a) shows distributions of factor of safety as a function of rising canal water elevations for the “water-filled gap” and the “ungapped” cases, and Figure 8.88(b) shows the resulting estimated distributions of the overall conjugate factor of safety for this west bank section.

Table 8.2 then presents the resulting estimated probabilities of failure vs. canal water elevation. The probability of failure at the actual peak Katrina water elevation of approximately + 10 feet (MSL) was low, but it was not negligible. Moreover, it would have increased rapidly with even minor additional increase in canal water level. The probability of failure becomes very high at the “design” water level of +12.5 feet (MSL).

It should also be noted that the marsh soils have likely been sheared (and thus softened) a bit, and that the overall strength of this section was therefore likely somewhat degraded by the loading it received during Katrina. Accordingly, it may not perform quite as well in subsequent loading in the future.

This levee and floodwall section protects the large population of the still undamaged Jefferson Parish. If the canal floodgate currently being installed, and future control of pumping, cannot guarantee that canal water levels will never exceed about Elev. +5 to 6 feet (MSL), then this section should be remediated.

8.3.8 The Breach Near the South End of the London Avenue Canal

A major breach occurred on the east bank, near the south end of the London Avenue Canal, as shown in Figures 8.1 and 8.2. Figure 8.89 shows an oblique aerial view of this breach under repair. The breach was approximately 80 feet in length, and it scoured to significant depth. Sands eroded and transported by the intruding floodwaters blanketed the neighborhood inboard of the breach to considerable depth over a surprisingly wide area, as shown for example in Figure 8.90.

Figures 8.91 and 8.92 show the floodwall sections at the south and north ends of the breach, respectively. In these photos it can be seen that these wall sections have not displaced (translated) laterally towards the inboard (“protected”) side; instead they have simply “dropped” into the hole eroded by the scour of the breach flow.

Clearance for the footprint of the levee and floodwall was very limited, and the neighboring homes and their back yards encroached closely on the levee. Levee maintenance was very poor along this section, and numerous large trees had been allowed to grow along the inboard toe. Many of these were actually rooted part way up the inboard slope face of the levee embankment itself, as shown in Figure 8.93 which is a view looking north from the breach location. These trees at the inboard toe represented an unacceptable risk as they can be blown over by storm winds, creating sudden voids that represent favorable paths for concentration of seepage flows and erosion in the critical toe area. Also, when they die the rotting root system can leave voids that can pose a significant hazard with regard to seepage and erosion in the critical inboard toe area.

Several large trees did topple at this site during Katrina, but in the absence of eyewitnesses it is not possible to be certain if they toppled before the breach, or as a result of erosion and scour after the breach opened. Figure 8.94 shows toppled trees at this site. Two large trees from the levee toe area within the breach footprint toppled during this event.

This breach was much shorter in length than the large breaches at the 17th Street Canal, the north end of the London Avenue Canal, and the southern breach on the IHNC at the west side of the Ninth Ward (each of which were hundreds of feet in length.) Instead, like the northern breach at the IHNC at the west end of the Ninth Ward, this was a narrow and deep breach; suggesting that underseepage rather than foundation instability may have been the key issue here.

As discussed previously in Chapter 4, the geology of the London Avenue canal differs significantly from that of the north end of the 17th Street Canal. The buried sand “ridge” runs laterally across the canal region, as shown in Figure 4.10 in Chapter 4, and relatively thick sand strata occur at shallow depths in the London Avenue Canal (and the south Orleans Canal) region. On the south side of this buried sand ridge, the sands tend to be dense as a result of wave action and energy from the Gulf side. On the lee side (the north side), the sands, especially at shallow depth, were protected and tend to be looser.

Figure 8.95 shows the locations of borings and CPT probes performed by the ILIT investigation at this site. Figure 8.96 shows a cross-section through the breach, based on our

own (ILIT) data as well as IPET data and data available prior to Katrina. The embankment has a modern (engineered fill) crown consisting of lightly compacted clay and silty clay, underlain by older fill of more variable composition. The embankment section rests atop variable “marsh” deposits consisting primarily of variably interbedded clays and organics. This “marsh” stratum is relatively thin, with a thickness of only 3 to 4 feet at the inboard toe, and it is underlain by about 2 to 3 feet of soft gray clay (CH).

This thin surficial marsh and clay “crust” is underlain by deep deposits of medium dense and then dense sands. In addition to the sheetpile curtain supporting the current concrete floodwall, there is an older sheetpile curtain on the outboard side that used to support a previous small floodwall at this location.

Strengths of the marsh deposits and the thin layer of underlying clay were determined based on the available data, and the resulting strength characterizations are summarized in the table within Figure 8.97, along with the estimated friction angles for the underlying sand units. Stability analyses showed high factors of safety with regard to “landslide type instability failure”, even for steady state seepage conditions at the maximum storm surge height of approximately Elev. +9 feet (MSL). Figure 8.106 shows the most critical potential slide surface for these worst case steady state seepage conditions. It was concluded that this breach was unlikely to have resulted from conventional foundation stability failure.

Numerous analyses of seepage were performed, varying the horizontal and vertical permeabilities of the various soil units and strata (in both the horizontal and vertical directions) over ranges considered reasonable for these soils. For all reasonable ranges of conditions, it was found the soils in the inboard toe area were vulnerable to erosion and potential piping at storm surge levels of less than Elev. +9 feet (MSL).

An example is shown in Figure 8.98, which shows the flownet and flow velocity vectors for a surge to Elev. +9 feet (MSL). Ranges of values of in situ permeability were modeled for the sandy strata (in transient flow analyses), and it was concluded for reasonable ranges of lateral permeabilities that nearly full equilibration of pore pressures (greater than 90 to 95% equilibration) at the inboard side levee toe region would occur within 30 minutes or less of outboard side canal water level rises. Given the rate at which the outboard side canal waters rose (see Figure 8.18), steady state seepage analyses were considered to provide an accurately (to slightly conservative) basis for assessment of underseepage pore pressures. The analysis shown in Figure 8.98 thus represents steady state flow conditions.

Figure 8.99 is a close-up from this figure showing localized conditions in the vicinity of the levee and floodwall. The sheetpiles are nowhere near deep enough to be effective in reducing massive underseepage flows through the pervious sands, and exit gradients near the inboard toe are unsafe with regard to erosion and the initiation of potential piping.

Figure 8.100 shows pore pressure contours from this same flow analysis. Hydraulic uplift forces at and just inboard of the toe exceed the weight of soil overburden, suggesting the possibility that hydraulic uplift ruptured the less pervious thin clay and marsh crust causing a “blowout” failure in this toe area.

Figure 8.101 shows hydraulic gradients for this same flow analysis. The exit gradients at the inboard toe are on the order of $i_o \approx 0.5$, representing a factor of safety with respect to erosion of approximately

$$FS = \gamma_b / (i_o \cdot \gamma_w)$$

where γ_b is the buoyant unit weight of soil, γ_w is the unit weight of water, and i_o is the exit gradient. For the lightweight marsh soils, with light buoyant unit weights, the calculated factor of safety is on the order of $FS \approx 0.8$ to 1.05 for the conditions shown in Figure 8.101. Any “bunching” or localized constriction of the flownet near the exiting face would further exacerbate the tendency to initiate erosion and the beginning of piping. Given the high variability of the thin surficial marsh deposits that “cap” this site, erosion and piping are highly under these conditions.

Figures 8.101 through 8.105 illustrate how such erosion can rapidly escalate as the flownet converges on even a slight void (Figure 8.102) to rapidly increase the localized exit gradient and accelerate the erosion process (as occurs progressively as the erosion enlarges the hole at the inboard toe in Figures 8.103 through 8.105.) This is actually a three-dimensional process, so the rate of acceleration of this erosion and “piping” process is actually more severe than can be properly illustrated in these two-dimensional figures.

Figure 8.107 is a schematic illustration of this process. As the flownet increasingly converges, and erosion continues to accelerate, and the erosion literally tries to “tunnel” back under the levee embankment. This produces slumping and periodic collapses into the opening void, and the process continues to accelerate until the crest is finally breached, at which point the intruding flows rapidly further scour the breach.

An additional possibility is that this type of erosion process may have been exacerbated by the toppling of a tree near the levee toe, as illustrated schematically in Figure 8.108. Flow towards the toe (and the trees rootball zone) weakens the ground and thus weakens the tree’s resistance to pullout failure under storm wind loading. Many trees toppled in this manner during the hurricane. If the tree near the toe topples, it created a large void toward which the exiting flownet would rapidly converge, initiating or greatly accelerating the type of erosion and piping process described above.

Figure 8.109 shows another view of this breach section, this time from the waterside and in late September of 2005. In this photo it can be clearly seen the breach is a very narrow feature, deeper at the north end (to the left in this view). On the inboard side our field team felt that the evidence suggested that the breach initiated either as a seepage erosion “blowout” or similar near the north end of the feature. There was a large tree that was uprooted at that location, but it could not be determined whether the tree fell before or after (as a result of) this failure and breach.

In the end, this breach scoured to significant depth and was then rapidly buried by the emergency embankment repair section, so there is no conclusive evidence left with which to determine which of the above described possible mechanisms (in detail) caused the actual

failure. It is apparent, however, that this failure was the result of underseepage and erosion of some form. The lack of sufficient sheetpile depth as to adequately reduce underseepage flows and toe exit gradients was an engineering lapse, and so was allowing the rampant growth of large trees in the inboard toe area.

The original design analyses for this section were performed by an outsourced engineering consultant, and were reviewed by the USACE (USACE; DM-19A.) In these analyses, the canal-side phreatic level was taken at the full design level (Elev. +12 feet, MSL), and the phreatic level at the inboard side levee toe was taken at Elev. -5 feet (MSL). Based on our investigation's transient flow analyses, for reasonable ranges of in situ lateral permeability, for the full (design) canal water elevation of +12 feet (MSL), the phreatic level at the inboard side levee toe due to underseepage would have actually been on the order of +2 to +5 feet (MSL). This represents a large increase in underseepage-induced uplift pore pressures and exit gradients, and is the principal difference between the pre-Katrina "design" analyses and our investigation's post-Katrina forensic analyses at this section.

8.3.9 The Breach and Distressed Sections Near the North End of the London Avenue Canal

An additional major breach occurred on the west bank near the north end of the London Avenue Canal, as shown in Figures 8.1 and 8.2. This too was a catastrophic breach as it rapidly scoured below mean sea level and so was one of the three large drainage canal breaches that continued to push water into downtown New Orleans for three days after Hurricane Katrina's passage.

Figure 8.110 shows an aerial view of the breach on the west bank. There was also a "distressed" section on the opposite side (on the east bank) that represents an incipient failure in progress; this failure was arrested in a partially developed state by the failure of the west bank section (which drew down the water level and thus saved the east bank.)

Figure 8.111 shows a view looking south along the canal, with the emergency repair embankment section on the west bank on the right, and the incipient failure section on the left side. If one looks closely, the floodwall on the left (east) side can be seen to be leaning away from the canal in this photo.

This was one of the most challenging sites for our investigation. Foundation soil conditions, and embankment and floodwall geometries, were similar on both sides of the canal. One side failed catastrophically, and the other appears to have begun to fail but to have been saved by the failure on the opposite bank. It was a challenge to develop a model that would predict the failure of the west bank before the east bank failure was able to fully develop. There are also a variety of data and evidence suggestive of a number of potential failure and distress modes evident at both sites (both sides of the canal), and sorting through these posed a significant challenge as well.

Figure 8.112 shows a view of the main breach on the west bank, taken from the south end of the breach on the outboard (water) side. In this photo it can be clearly seen that the water-side toe section of the earthen embankment is still in place, and that the

floodwall/sheetpile curtain and the inboard side of the earthen levee have been separated from it and pushed to the inboard side.

Figure 8.113 shows conditions at the inboard toe of the failed embankment section on the west shoreline. The small clubhouse shown had originally been at the same elevation as the nearly adjacent house, but was lifted nearly 7 feet vertically by the displacements during the failure. Some initial field investigators suggested that this was evidence of rotational movement, but our investigation found that this clubhouse (and the ground upon which it stood) was raised vertically by heave due to “plowing” as the main levee embankment displaced laterally (without rotation.) The confined uplift region, and its “humped” nature, are clearly evident beneath the small clubhouse in this photo.

Figure 8.114 shows a view of the inboard toe of the “distressed” (displaced) embankment and floodwall section on the east shoreline, taken on the outboard (water) side. As shown in this photo, the concrete floodwall leaned away from the canal, and a gap with a maximum width of 2.5 feet (and a common width of 1.5 to 6 feet) opened between the outboard side of the earthen levee embankment and the concrete floodwall (and its supporting sheetpile curtain.)

Figure 8.115 shows the other side of this same floodwall section. As shown in this photo, the displaced floodwall leaned to the inboard with a readily discernable tilt of up to 8°. The next photo, Figure 8.116, shows conditions along the inboard base of the floodwall (at the feet of the photographer who took the photo of Figure 8.115.) A series of apparent “sinkholes” occurred along the inboard side contact between the concrete floodwall and the crest of the earthen levee at this location.

Figure 8.117 shows conditions at the inboard toe immediately below the sinkholes of Figure 8.116. A prominent sand boil feature, with sandy ejecta, occurred at this location. Less apparent, but important, was the hummocky wrinkling of the nearly level ground inboard of the toe of the levee, and the slight overthrust feature adjacent to the sand boil. This overthrust feature was apparently missed by many field investigators, but our team noted it and went back and excavated it during our subsequent field boring, sampling and CPT program and found that it was indeed the toe thrust of the beginning of a translational instability feature.

Figure 8.118(a) shows a cross-section through the west side breach prior to Katrina, and Figure 8.118(b) shows this same section after the failure. The failure on the west side was a translational failure of the embankment, sliding along the interface between the foundation sands and the overlying less pervious layer of silty clay (CL/ML).

Figure 8.119(a) shows a cross-section through the east side “distressed” section prior to Katrina, and Figure 8.119(b) shows this same section after the hurricane. The displacement and tilting of the floodwall was the result of the initiation of slippage, once again at the interface between the foundation sands and the overlying less pervious layer of silty clay. Unlike the west bank, this slippage progressed only enough to produce displacements of approximately 1.5 to 2.5 feet, whereupon these movements were arrested as the failure and breaching on the opposite bank rapidly drew down the canal water level and reduced the lateral push against the sheetpile curtain and floodwall.

Figure 8.120 shows a plan view of both sides of the canal, indicating the locations of the borings and CPT performed as part of this investigation.

Figure 8.121 shows the longitudinal subsurface soil profile developed along this section of levee on the west bank during the original design studies, and Figure 8.122 shows the re-interpretation of this section by this study team based on the original boring data. Figures 8.123 and 8.124 show the same pairing of profiles for the east bank side.

Processing of the available geotechnical data was performed using essentially the same methods and procedures as were described in detail in the preceding sections, and much of the detail will be omitted here in the interest of brevity.

Figures 8.125 and 8.126 show the best estimated profiles of strength vs. depth and Su/P vs depth on the west bank (breach) side for profiles (a) beneath the full levee embankment overburden, and (b) inboard of the levee toe.

Figure 8.127 shows estimated friction angles across the transition from the base of the silty clay stratum (CL/ML) into the underlying clayey sands and sands. Friction angles were estimated from the CPT data using two correlations, and they were also estimated based on the SPT data available from the borings. Also shown are the results of two direct shear tests performed on “undisturbed” samples as part of these studies.

Figure 8.109(a) shows an “undisturbed” sample from the transition across the silty clay into the underlying sands. As shown in this figure, this transition was semi-gradational rather than abrupt. The base of the silty clay layer is underlain by fine clayey sands with variable fines content. Near the contact the fines content is high enough that the clayey fines dominate the shear strength behavior. The fines content rapidly decreases over the next 6 inches or so, and eventually the fines content of the remainder of the layer remains relatively stable at between 5% to 10%. The green line in this figure represents our best estimate of the approximate operative effective friction angle through this zone.

It was not possible to discern with certainty the elevation to which pore pressures arising from underseepage passing beneath the sheetpile curtain through the more open, pervious sands at depth due to the transient rising storm surge penetrated (vertically) upwards into this transition zone. Accordingly, various combinations of partial pore pressure development may be postulated at different elevations across this transition, and these may be paired with various effective friction angles to evaluate the shear strength within this narrow, and critical zone.

Several combinations were postulated and analyzed in these studies. Higher (more completely penetrating) pore pressures more nearly approaching steady state flow are clearly appropriate at the base of this transition zone, and these would be paired with friction angles on the order of $\phi \approx 30$ to 32° . A few inches higher in the transition zone the effective friction angle would be somewhat lower, but this would be offset by reduced penetration of pore pressures, resulting in largely similar estimates of resultant frictional shear strength. In the end, an effective friction angle of 31° was selected, and this was coupled with assumed rapid

development of steady state pore pressures as the storm surge rose. (For reasonable ranges of in situ permeability of the deeper, more open and pervious sands and with reasonable ranges of specific storage for these initially saturated deposits; pore pressure development at the inboard side toe region within the pervious deeper sands was approximately 65 to 90% developed within two hours of outboard side (canal) water level increases.)

Figures 8.128 through 8.130 show the same sequence of figures, this time for conditions on the east bank (distressed) side of the canal. Once again the transition between the silty clay and the underlying clayey sand is the critical region. As with the west bank, an effective friction angle of 31° was selected for analysis, and this was coupled with assumed rapid development of full steady state underseepage as the storm surge rose within the canal.

Figure 8.131 shows the analysis cross-section and principal soil properties modeled for analysis of the west bank breach site. Analyses were performed using both finite element analysis methods (again using the program PLAXIS) and limit equilibrium methods (Spencer's Method).

Figure 8.132 shows normalized shear strain contours for the west bank (breach) section at a storm surge level of Elev. +9 feet (MSL). Gapping initiated at the outboard side of the floodwall and its supporting sheetpile curtain initiated in this analysis at a canal surge elevation of between +7 to +8 feet (MSL), and was fully developed by a surge elevation of +9 feet, as shown in this figure.

Figure 8.133 shows normalized shear strain contours for the east bank (distressed) section, this time for a slightly higher surge to elevation +10 feet (MSL). This the upper bound estimate of the surge elevations achieved during Katrina. Gapping developed in this east bank section at a surge elevation of between +7 to +8 feet, and was fully developed by a surge elevation of +9.5 feet (MSL).

These conditions produce a predicted failure of the west (breach) side at a surge elevation of approximately +9.5 feet (MSL) in these PLAXIS analyses, and the east side displaces a bit (with associated lateral displacement and tipping of the floodwall) but remains barely stable to a surge elevation of +10 feet (MSL).

Figures 8.134 and 8.135 show a simultaneous analysis of both sides of the canal, and the predicted ("best estimated" properties and flow) conditions for a storm surge to elevation +9 feet (MSL). Figure 8.134 shows normalized shear strain contours, and Figure 8.135 shows the associated predicted deformations and displacements. The west side has failed catastrophically, and the east side section is "distressed" (with lateral displacements of approximately 2 to 3 feet and some tilting of the floodwall. This closely matches the field observations.

Figure 8.136 shows the associated PLAXIS-based prediction of the critical path to failure for each side of the canal. Once gapping occurs, the extra "push" of the water in the gap is sufficient to destabilize the west bank at a surge height of approximately +9 to +9.5 feet (MSL), but the east bank section remains barely stable until a surge height of +10 feet (the upper bound of the estimated surge height that actually occurred).

Figures 8.137 through 8.159 repeat these same analyses, this time using classic seepage analyses to predict pore pressures and gradients resulting from the underseepage flows as the storm surge rises, and limit equilibrium analyses (Spencer's Method) coupled with these predicted pore pressure and gradient conditions to evaluate overall stability for both sides of the embankment. Once again, rapid development of essentially full steady state underseepage was assumed, and an effective friction angle of 31° was modeled at the interface between the silty clay and the underlying clayey sand.

Figures 8.142 and 8.146 show the most critical failure surfaces on the west bank (breach) side for a surge elevation of +9 feet (MSL), with and without gapping respectively. Figures 8.153 and 8.157 show the same two cases for the east bank (distressed) side, again for a surge height of +9 feet (MSL).

Figure 8.159 summarizes the results of these limit equilibrium analyses for both sides of the canal, and the heavy red lines show the estimated most critical paths to failure. Once again the west bank side fails at a surge height of slightly less than +9 feet, but the east bank (distressed) side remains barely stable at this surge elevation. The blue horizontal dashed line in this figure represents our investigation team's best estimated surge elevation in the canal at the time of the breach and failure of the west bank section.

These analyses show that the observed behaviors were not the result of underseepage and resultant piping erosion. The behaviors on both sides of the canal were, instead, the result of lateral translational instability (and incipient instability), with the critical potential failure mode on both banks being lateral translational sliding on the interface between the silty clay and the underlying clayey sands. This sliding was made possible by the high porewater pressures in the foundation soils at and near the base of the inboard-side levee toe due to underseepage.

This exactly fits with the observed field data. The "sinkholes" at the crest of the embankment on the east side were the result of tilting of the slightly displaced floodwall, and the resulting opening of a gap between the floodwall and the embankment into which embankment soils could fall. This correlates with the observation the "sinkhole features" were all narrow, and were all parallel and adjacent to the floodwall (see Figures 8.114 and 8.117.)

8.3.10 Summary and Findings

A large number of critical errors and poor judgements jointly contributed to the catastrophic failures that occurred along the drainage canals. There were conceptual errors in the layout and fundamental design of the levees and floodwalls, there were policy and funding issues that greatly reduced the level of safety of the overall system, and there were engineering errors in the analysis and design of individual sections.

No one organization, agency or group of individuals had a monopoly on their contribution to this disaster. Federal government (including the Congress), the Corps of

Engineers, local government and local oversight agencies (including the local Levee Board and the local Water and Sewerage Board), and outsourced engineering firms all contributed.

The resulting system failed catastrophically, and at multiple locations. And it failed at significantly less than the intended levels of “design” (storm surge) loading. Moreover, it is clear that additional sections were saved from failure only by the catastrophic failures of nearby breaches, which drew down the water levels and so reduced the loading on additional potentially unstable levee and floodwall sections.

The results of these failures were catastrophic. The vast majority (approximately 80%) of the eventual floodwaters that flowed into the main Orleans East Bank (downtown) protected area came through the breaches in the drainage canals. These flows overfilled the sub-basin north of the Metairie Ridge, and then crossed this ridge and flowed into the southern areas as well where they greatly exacerbated flooding that had already occurred as a result of overtopping and failures of levees and floodwalls along the west side of the IHNC. In the absence of the drainage canal failures there would still have been localized flooding and damage near the IHNC, but this would have been minor relative to the eventual damages that resulted when the canal breaches filled a majority of the overall basin.

The localized flooding near the IHNC would have posed relatively little threat of loss of life; the damages would have been (relatively) limited and the floodwaters could have been pumped out in a matter of days. Instead, roughly half of the 1,293 fatalities (to date) attributed to flooding of the New Orleans region occurred in the Orleans East Bank (downtown) protected basin, and a roughly similar fraction of the devastating regional economic damages as well.

The following is a listing of critical errors and poor judgements and decisions that contributed significantly to the poor performance of the drainage canal levees and floodwalls during Hurricane Katrina:

1. The decision not to install floodgates at the north ends of the three drainage canals to prevent uncontrolled water level rise due to storm surge within the canals was largely the result of poor interaction between the local Levee Board and the local Water and Sewerage Board, and their inability to resolve their differences in the interests of the greater Public good (and safety). Lawsuits by environmentalists against this system also worked against the floodgates. As a result, the canals remained open to storm surges; essentially inviting the enemy (storm surge) into a poorly protected section of the interior of the protected ring around metropolitan New Orleans.
2. The decision not to purchase additional land (right of way) to permit widening of the levees required that the system be extended vertically without allowing provision of additional levee width and mass with which to resist the increased floodwater forces associated with the increased height. Short-term savings here resulted in tens of billions of dollars in losses.
3. Similarly, the failure to garner access and control of property at the inboard (protected side) toe of the levees prevented full and proper inspection of this critical area. It also

led to unacceptable risk associated with growth of trees on the inboard side levee slopes and toes, and the literal undermining of levee toes by excavation of in-ground swimming pools in this critical inboard toe area.

4. The designers failed to take advantage of critical lessons from an expensive and well-directed research program that involved construction of a full-scale model levee and floodwall on nearly identical foundation soils in the nearby Atchafalya basin. This model was loaded to failure, and the failure mode observed involved opening of a gap on the outboard side of the floodwall, water entering into the gap, and subsequent pressures on the floodwall and sheetpiles pushing the inboard side section of the earthen embankment sideways (the “cut the cake in half and slide it” failure mode). This failure mode was neglected in the subsequent design of the levees and floodwalls lining the canals, and at least two of the catastrophic failures (breaches), and two additional “incipient” failures were the result of this failure mechanism.
5. The designers also failed to take account of the influence of stress history and effective overburden stresses on the strengths of the foundation soils beneath a number of the embankments. Furthermore, they deviated from USACE policy by using average shear strengths (not strengths slightly lesser than the average data), and by “averaging” strengths across lateral distances that were too large. These errors and shortcomings in the determination and selection of soil shear strength parameters played a critical role in the catastrophic failure of the east bank near the north end of the 17th Street canal.
6. Optimistic assumptions, and misinterpretation of two field tests, led to the assumption that system permeability was low enough that underseepage would not be a critical issue during “transient” (short-term) rises in canal water levels during hurricane induced storm surges. This was a critical error, and it resulted in inadequate sheetpile lengths throughout the drainage canals (especially the London Avenue Canal), and along the IHNC. These sheetpile curtains routinely extend to insufficient depths as to adequately “cut off” underseepage flows, and the resulting underseepage flows were principal contributors to the catastrophic failures observed at both of the major breach sections on the London Avenue canal. These inadequate cut-offs continue to be a potentially critical issue at other sections that did not (yet) breach during hurricane Katrina, and they appear to have been a principal factor in the two massive breaches on the east bank of the IHNC (at the edge of the Lower Ninth Ward; see Chapter 6) as well.
7. Insufficient site investigation was performed for the design of these critical systems protecting a major metropolitan population. Given the difficult and complex foundation soil conditions, additional borings and testing would have represented a very modest incremental expenditure, and would have greatly improved the information available as a basis for analysis and design of these important sections.
8. Errors and poor judgements were made in engineering analysis and design of these sections. Soil properties were extrapolated laterally over inappropriately large distances, and without adjustment for the resulting uncertainties. Archaic analysis

techniques were employed (the Method of Planes), and project-specific research (a full scale test embankment and floodwall in the nearby Atchafalya basin) was ignored, resulting in failure to analyze the failure mode (“cut the cake in half and slide it”) that proved critical for at least two of the catastrophic drainage canal breaches, and likely also for the two massive breaches at the east side of the IHNC (adjacent to the Ninth Ward.) This mode was also evident at an additional “incipient” failure section on the London Avenue canal that was saved from failure, by the failure of the even weaker section on the opposite shoreline (which immediately drew down the local water levels.) The stability of the entire canal system should be considered potentially suspect until it can be properly re-evaluated with regard to this potentially critical mechanism.

9. Design review was inadequate. Errors and questionable judgements that would have been expected to be caught and challenged by a properly convened independent external review panel went unchallenged. On one occasion when reviewers from the USACE Division level in Vicksburg did catch and challenge such issues, they were rebuffed by the local District Chief who declared those issues to be a matter of “judgement”.
10. The Factor of Safety (FS) used for design of these vital levees and floodwalls was set at only $FS = 1.3$ for the case of “transient” storm surge loading. As discussed in detail in Section 8.3.7.1 this is inappropriately low for systems critical for the safety of large populations, and for the difficult and challenging foundation soils conditions of the region. This issue is discussed in significantly more detail in Chapters 11 and 12.
11. Congressional funding (appropriations) were problematic. Funding was irregular and somewhat unpredictable, representing a difficult basis for design and construction of a system intended to be contiguous (seamless) and to protect a large metropolitan population. Strategic decisions, and conceptual design, were often driven by a need for frugality. In addition, when appropriations did arrive, some elements of the system had to be further streamlined for economy. The relatively minor savings achieved now pale in comparison to the many tens of billions of dollars in losses that ensued.
12. Pace of funding was also problematic. At the time of Katrina’s arrival, the flood protection system in the canal district was still incomplete.... fully 51 years after the flooding from Hurricane Betsy that inspired the inception of construction of the improved flood protection system. Three of the bridges across the drainage canals still had not yet had their side walls raised, so three “holes” remained in an otherwise contiguous system. These “holes” at the bridges were not critical during Hurricane Katrina only because: (a) the storm surge was less than the full design load case, and (b) catastrophic nearby breaches (failures) occurred. (An additional “hole” in the system, at the south end of the Orleans Canal, was yet another result of dysfunctional interactions between the local levee board and the local water and sewerage board (as discussed previously in item #1 above.)

Many of the issues above, from conceptual design issues through engineering analysis details and even selection of appropriate Factors of Safety would have been expected to be

challenged by a properly convened independent review panel. Unfortunately, in the current system with myriad local interests and no strong local entity able to convene appropriate levels of unbiased expert review capability, this critical element was absent during the design and construction of these important flood protection system elements.

In addition, there was a lack of centralized authority, and of clear areas of responsibility. Involvement of a significant “local” institutional presence of significant stature and resources was lacking. The local levee board lacked the resources and funding to mount serious review of the Federal plans and designs, and the mandate to challenge problems that should have been apparent at early stages.

In the end, the performance of the flood protection system along the three drainage canals was unacceptable, and resulted in catastrophic loss of life and property throughout a major metropolitan region.

8.4 References

- Burk & Associates, Inc (1986) *Geotechnical Investigation of London Avenue Outfall Canal*, OLB Project No 2049-0269, Vol I and II, prepared for the Board of Levee Commissioners of the Orleans Levee District, New Orleans, Louisiana.
- Foott, R and Ladd, C.C (1977) “Behavior of Atchafalaya levees during construction”, *Geotechnique*, 27, No2, pp 137-160.
- GEO-SLOPE/W (2004) “Complete Set of Manuals”, John Krahn (Edit.), Calgary, Alberta, Canada.
- IPET – Interagency Performance Evaluation Task Force (2006) *Performance Evaluation Status and Interim Results*, Report 1 of a Series, Performance Evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System, January 10, US Army Corps of Engineers.
- IPET – Interagency Performance Evaluation Task Force (2006) *Performance Evaluation Status and Interim Results*, Report 2 of a Series, Performance Evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System, March 10, US Army Corps of Engineers.
- Jackson, R. B., (1988), “E99 Sheet Pile Wall Field Load Test Report”, Technical report No.1, US Army Engineer Division, Lower Mississippi Valley, Vicksburg, MS
- Jamiolkoski, M, Ladd, C.C., Germaine, J.T., and Lancellotta, R. (1985) “New developments in field and laboratory testing of soils.” *Proc. 11th International Conf. on Soil Mechanics and Foundation Eng.*, San Francisco, 1, 57-154.
- Karlsrud, K., Lunne, T and Brattlien (1996) “Improved CPTU interpretations based on block samples”, *Nordic Geotechnical Conference*, 12 Reykjavik 1996. *Proc*, Vol.1, pp 195-201.

- Kaufman, R and Weaver, F (1967) "Stability of Atchafalaya Levees", Journal of the Soil Mechanics and Foundations Division, ASCE, SM 4, pp 157 -176.
- Kemp, P., and Mashriqui, H. (2006). Personal Communication
- Lacasse, S., Ladd, C.C., and Baligh, M.M. (1978) "Evaluation of field vane, Dutch cone penetrometer and piezometer probe testing devices." Res. Report R78-26, Dept. of Civil Eng., MIT, 375 pp.
- Ladd, C.C and DeGroot, D. 2003. "Recommended Practice for Soft Ground Site Characterization", Arthur Casagrande Lecture, 12th Panamerican Conference on Soil Mechanics and Geotechnical Engineering, June, Boston.
- Lunne, T., Lacasse, S., and Rad, N.S., (1989) "General report: SPT, CPT, PMT, and recent developments in in-situ testing", Proceedings, 12th International Conference on soilmechanics and foundation engineering, Vol.4, Rio de Janeiro, pp. 2339-2403
- Mashriqui, H., (2006), Personal Communication
- Mayne, P.W and Mitchell, J.K (1988) "Profiling of Overconsolidation Ratio in Clays by Field Vane", Canadian Geotechnical Journal, Vol 25, No 1, February, pp 150-157.
- Oner, M., Dawkins, W.P., and Hallal, I., (1997) "Soil Structure Interaction Effects in Floodwalls", Electronic Journal of Geotechnical Engineering
- Oner, M., Dawkins, W.P., and Mosher, R., (1997) "Shearing Method for Soil Structure Interaction Analysis in Floodwalls", Electronic Journal of Geotechnical Engineering
- Robertson, R.K, and Campanella, R.G. (1983) "Interpretation of Cone Penetration Tests. Part I: Sand", Canadian Geotechnical Journal, Vol 20, No 4, Nov, pp 718-733.
- Seed, R.B. (2005) "CE 270B Class Notes", University of California, Berkeley.
- PLAXIS Finite Element Code for Soil and Rock Analyses (2004). "Complete Set of Manuals", V8.2 , Brinkgreve yVermeer (Edit.), Balkema, Rotterdam, Brookfield.
- USCE (1989) London Avenue Outfall Canal, Design Memorandum No 19A General Design, Vol 1 and Vol 2, Louisiana, Department of the Army, New Orleans District, Corps of Engineers, New Orleans, Louisiana.
- USCE (1990) Orleans Parish (Jefferson Parish) 17th St. Outfall Canal, Metairie Relief, Design Memorandum No 20 General Design, Vol 1 and Vol 2, Louisiana, Department of the Army, New Orleans District, Corps of Engineers, New Orleans, Louisiana.
- Van Heerden, I., (2006), Personal Communication

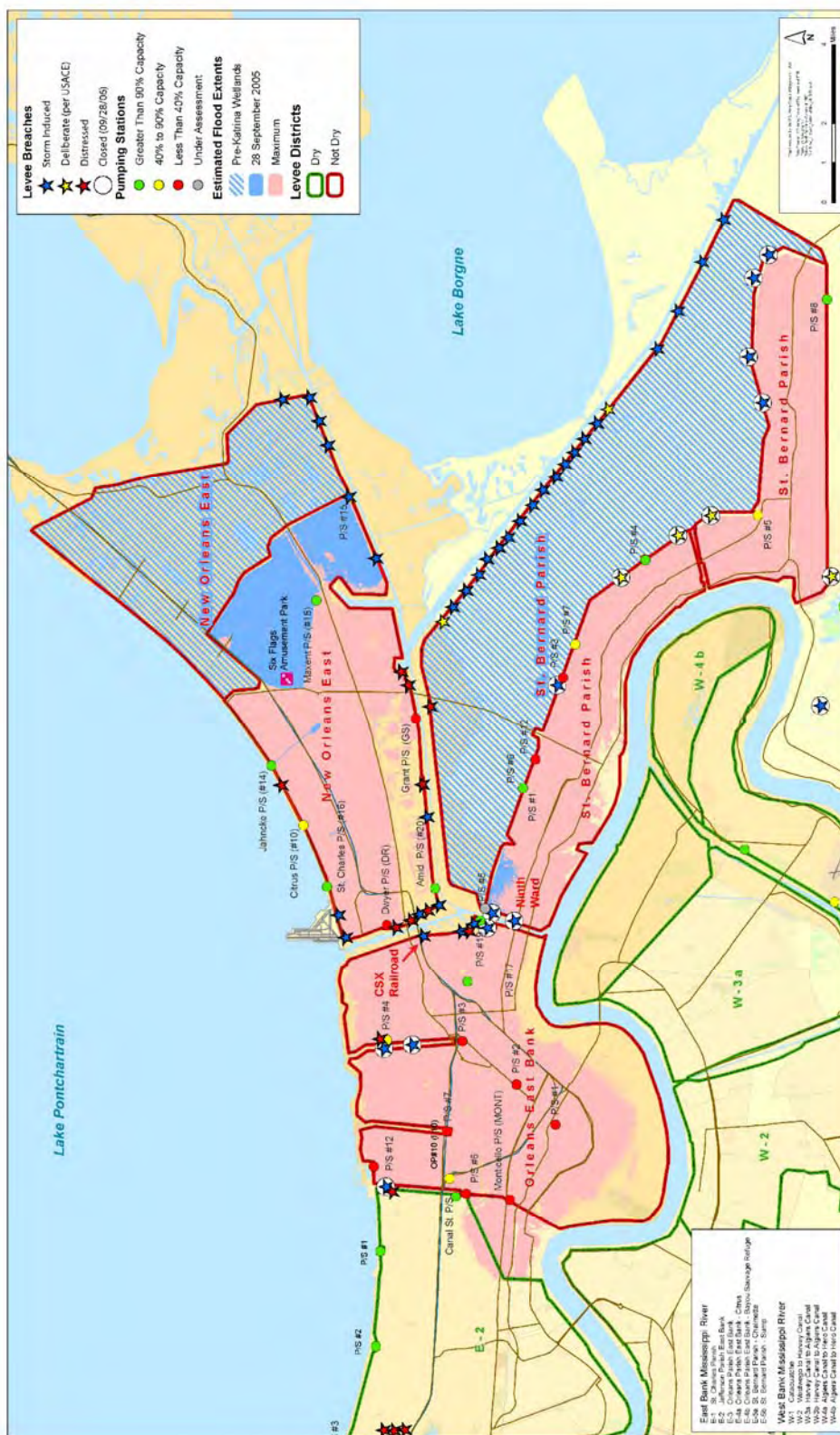


Figure 8.1: Map showing principal features of the main flood protection rings or “protected areas” in the New Orleans area.
[Modified after USACE, 2005]

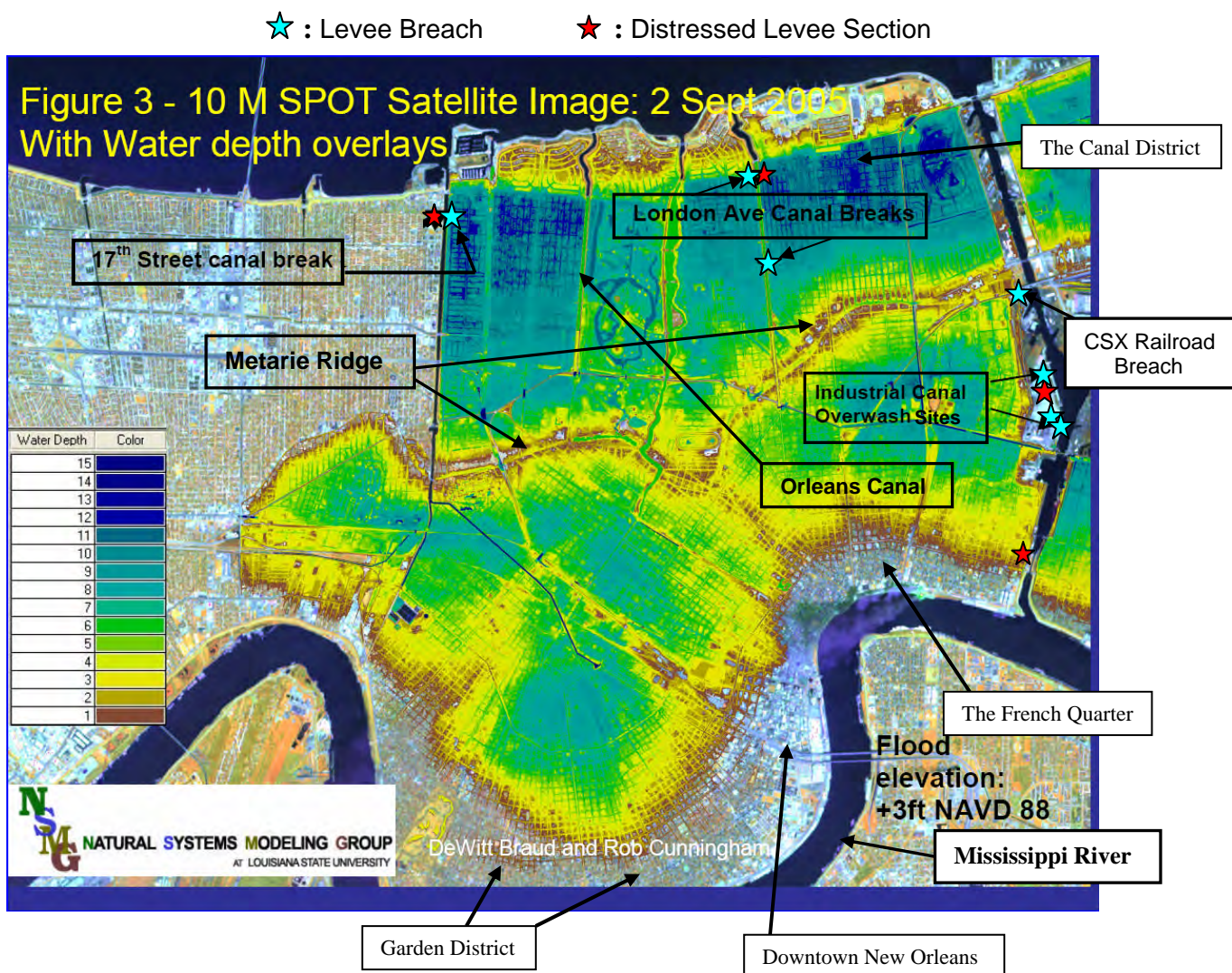
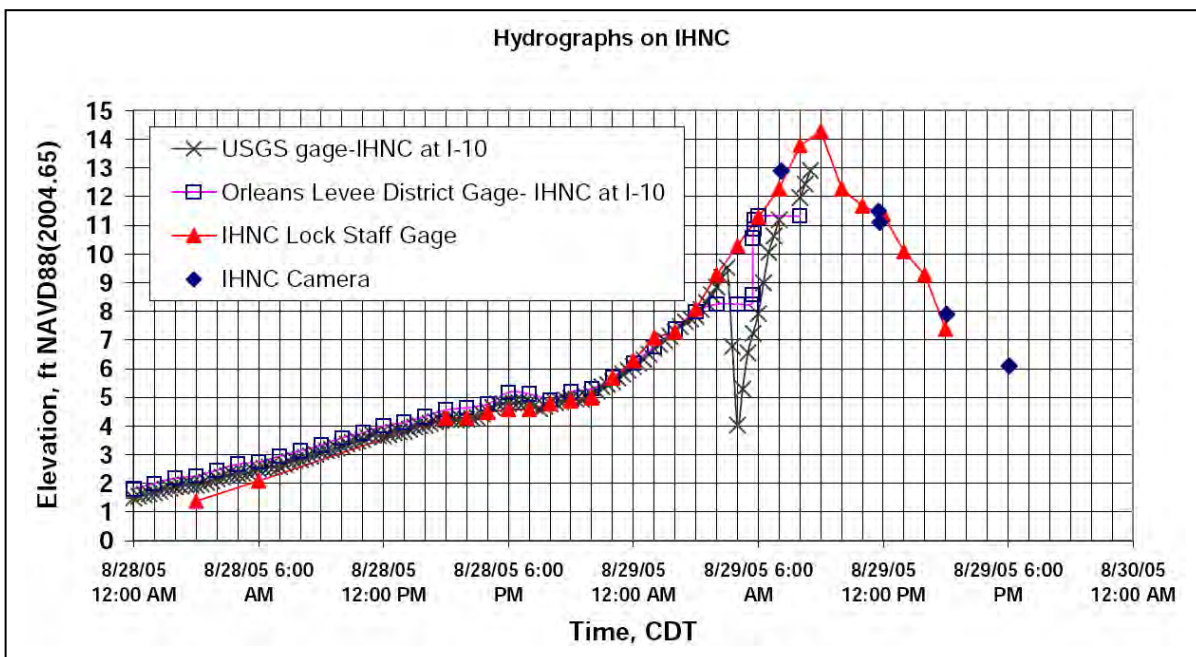


Figure 8.2: Plan View of the Orleans East Bank (Metro) protected area showing approximate depth of flooding on September 2, 2005.

[Modified after Mashriqui, 2006]



Source: IPET Interim Report No. 2; April, 2006

Figure 8.3: Hydrographs showing measured (and photographed) water levels at gage stations along the Inner Harbor Navigation Channel (IHNC).



Photograph by Rune Storesund

Figure 8.4: Concrete storm gate at the west bank of the CSX Railroad crossing of the IHNC, after Hurricane Katrina, showing the steel floodgate missing.



Photograph by Rune Storesund

Figure 8.5: View from the inboard side of the CSX Railroad breach site from Figure 8.4 showing scour of the roadway fill adjacent to the railway embankment fill.



Photograph by Rune Storesund

Figure 8.6: The I-wall section breach behind the Port of New Orleans, showing floodwall failure.



Photograph by Rune Storesund

Figure 8.7: Second view of the I-wall failure at the Port of New Orleans showing the trench scoured by overtopping flows.

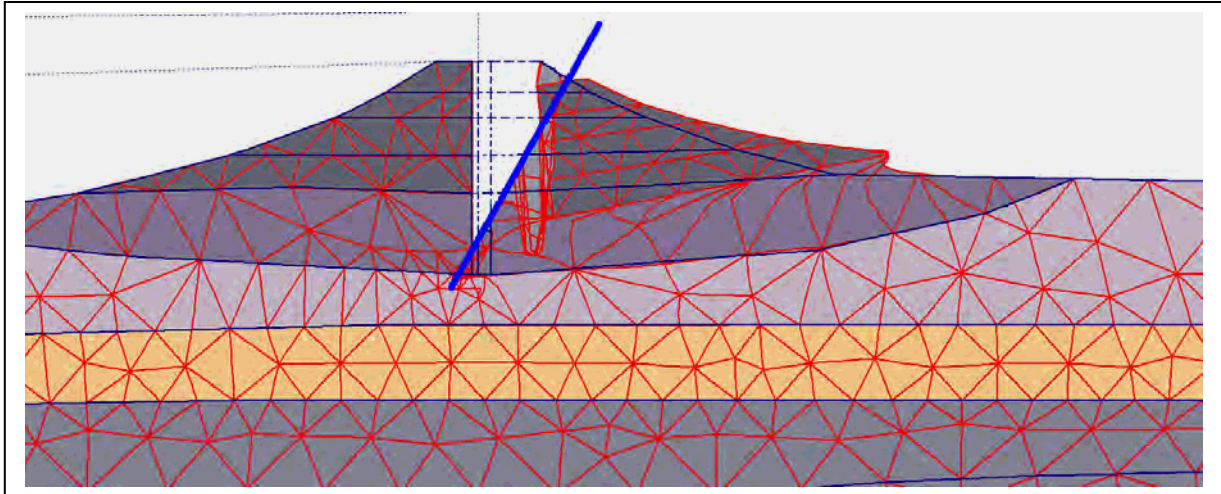


Figure 8.8: Deformed mesh of I-wall failure at the IHNC behind the Port of New Orleans.

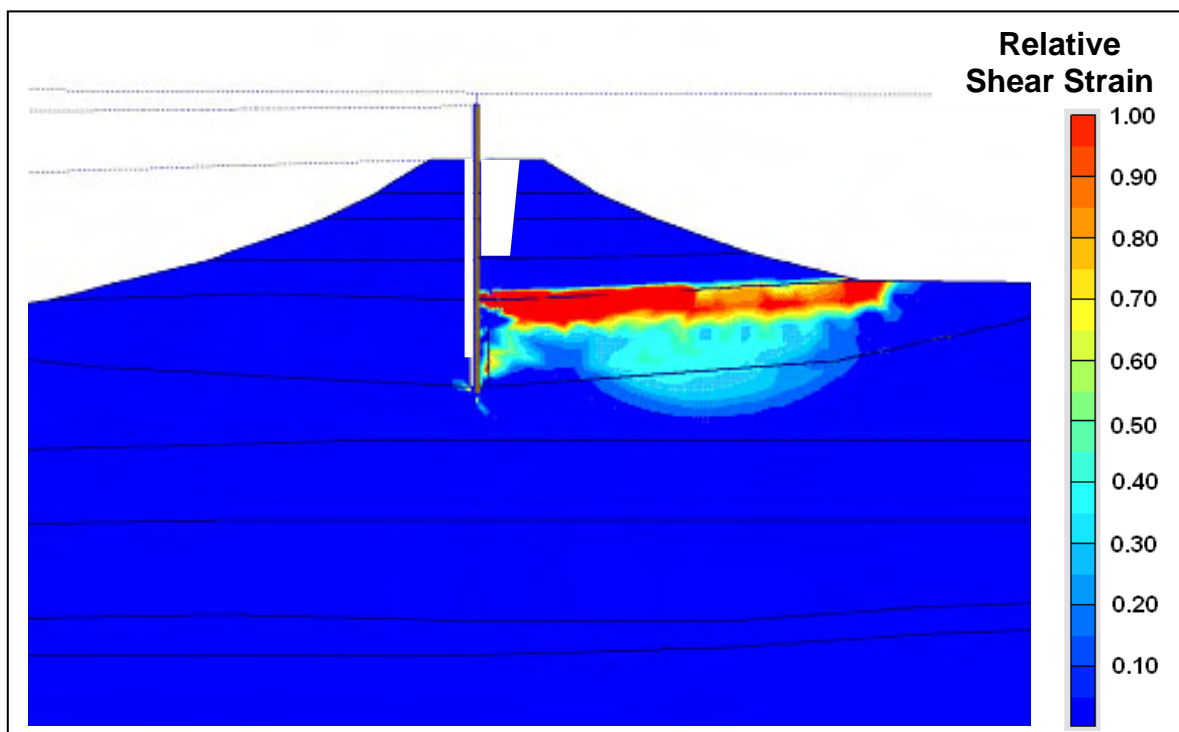


Figure 8.9: Finite element analysis of I-wall failure at the IHNC behind the Port of New Orleans, showing shear strain contours at point of wall failure.

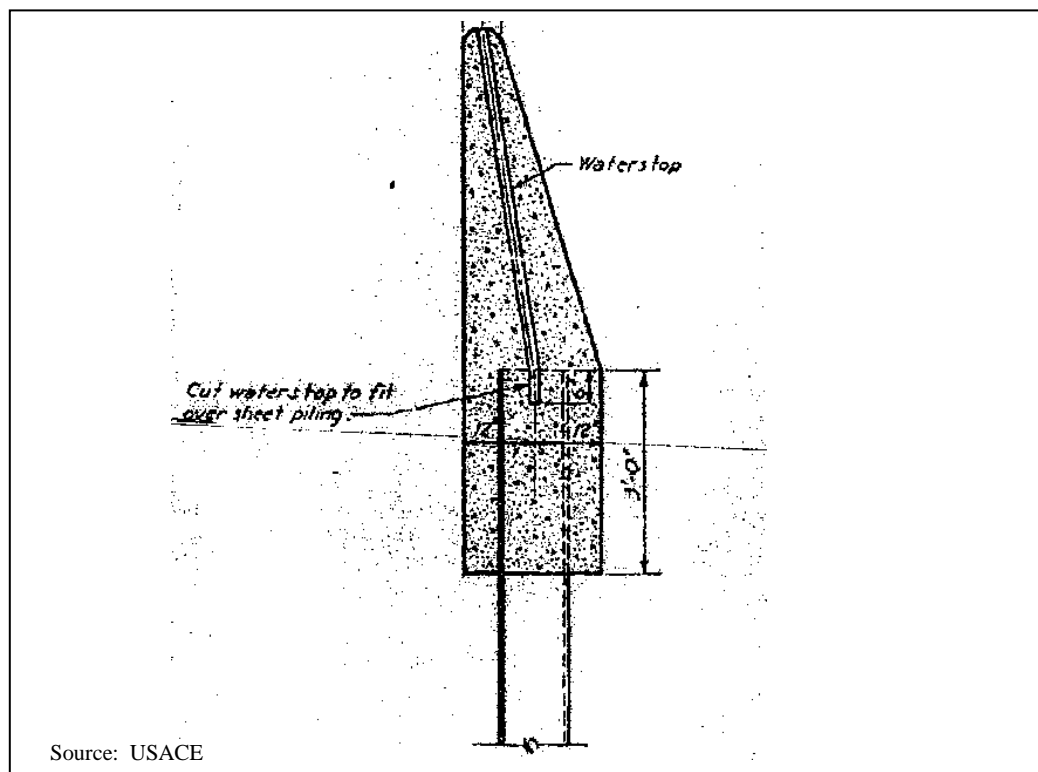


Figure 8.10(a): Typical reinforced concrete I-wall atop a sheetpile curtain.

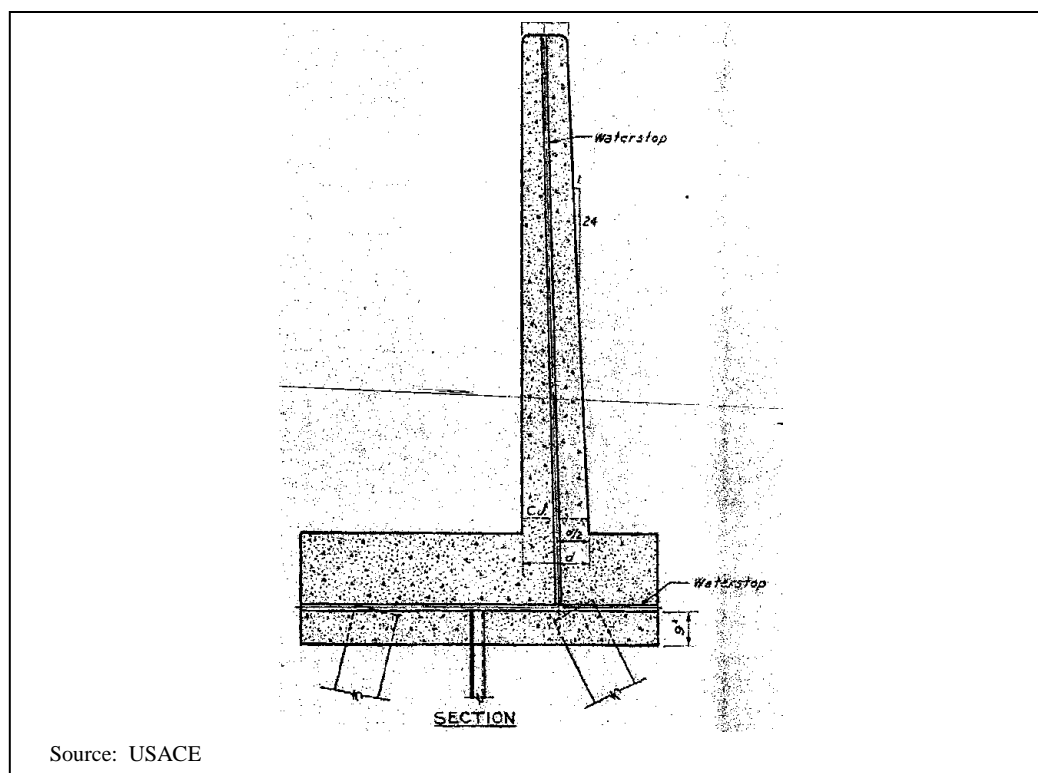


Figure 8.10(b): Typical reinforced concrete T-wall atop a sheetpile curtain.



Photograph by Rune Storesund

Figure 8.11: Concrete floodwall and steel floodgate structure at railroad yard behind the Port of New Orleans.



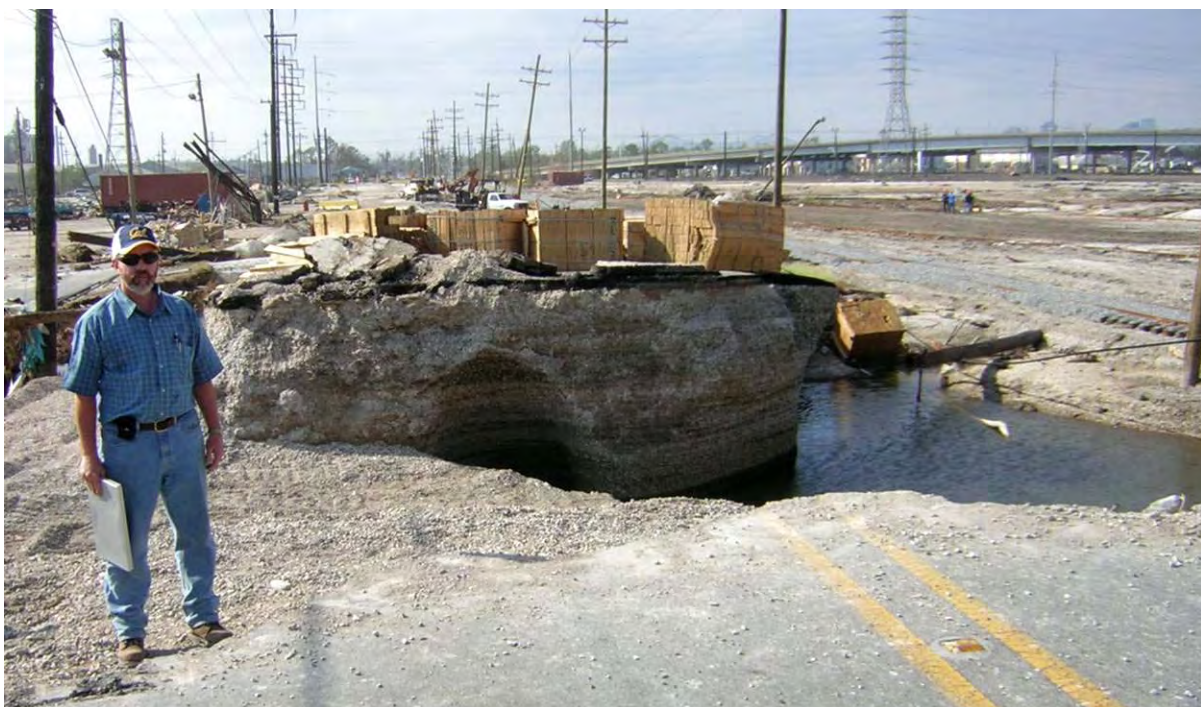
Photograph by Rune Storesund

Figure 8.12: Erosion, gapping and offset of adjacent wall sections at the east end “transition” of the concrete floodwall and gate structure from Figure 8.11.



Photograph by Rune Storesund

Figure 8.13: Erosional breach behind the southern end of the Port of New Orleans at contact between embankment section and structural concrete monolith.



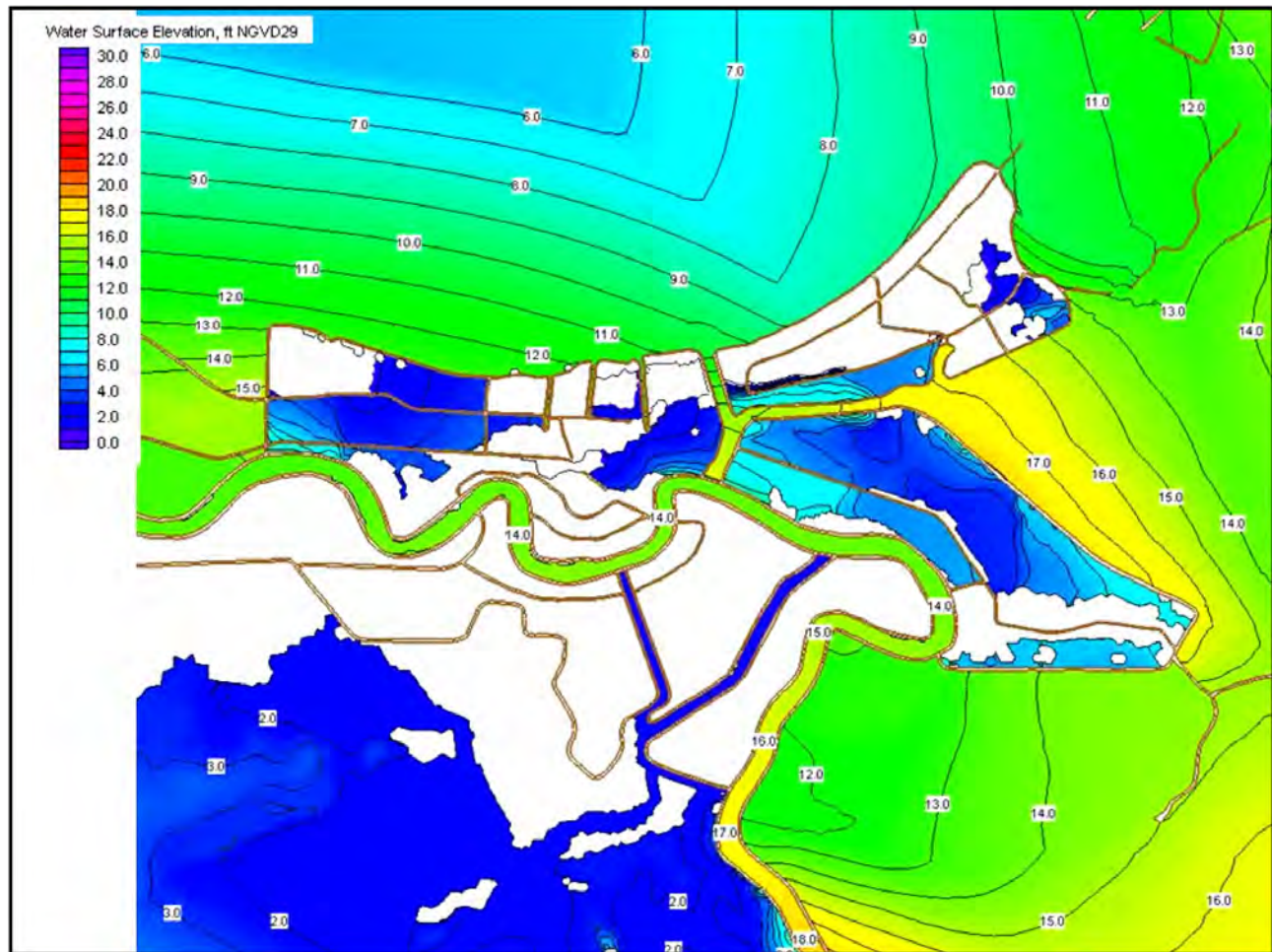
Photograph by Rune Storesund

Figure 8.14: Close-up view of the highly erodeable shell sand fill at the breach shown above in Figure 8.12.



Photograph by Rune Storesund

Figure 8.15: View of second large erosional breach behind the southern end of the Port of New Orleans.



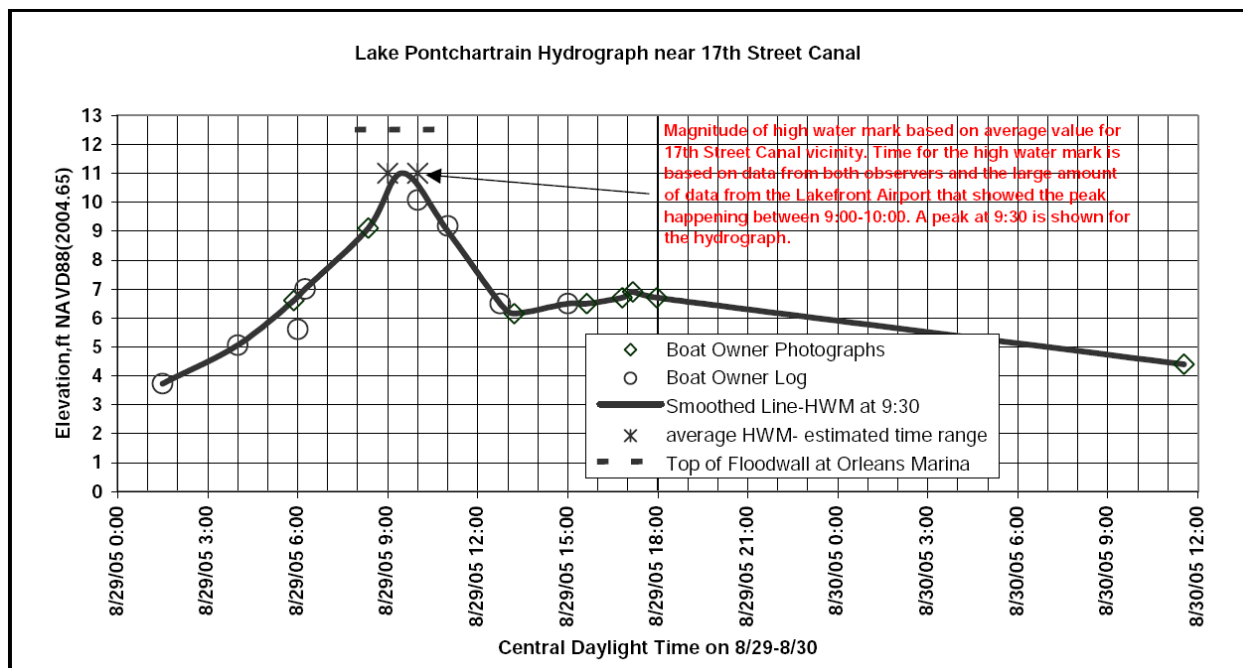
Source: IPET Report No. 2, April, 2006

Figure 8.16: Approximate contours of maximum storm surge heights along the southern end of Lake Pontchartrain. [Note: Elevations in Feet, NGVD29]



Source: IPET Report No. 2; April, 2006

Figure 8.17: High water marks near the mouth of the 17th Street Canal.



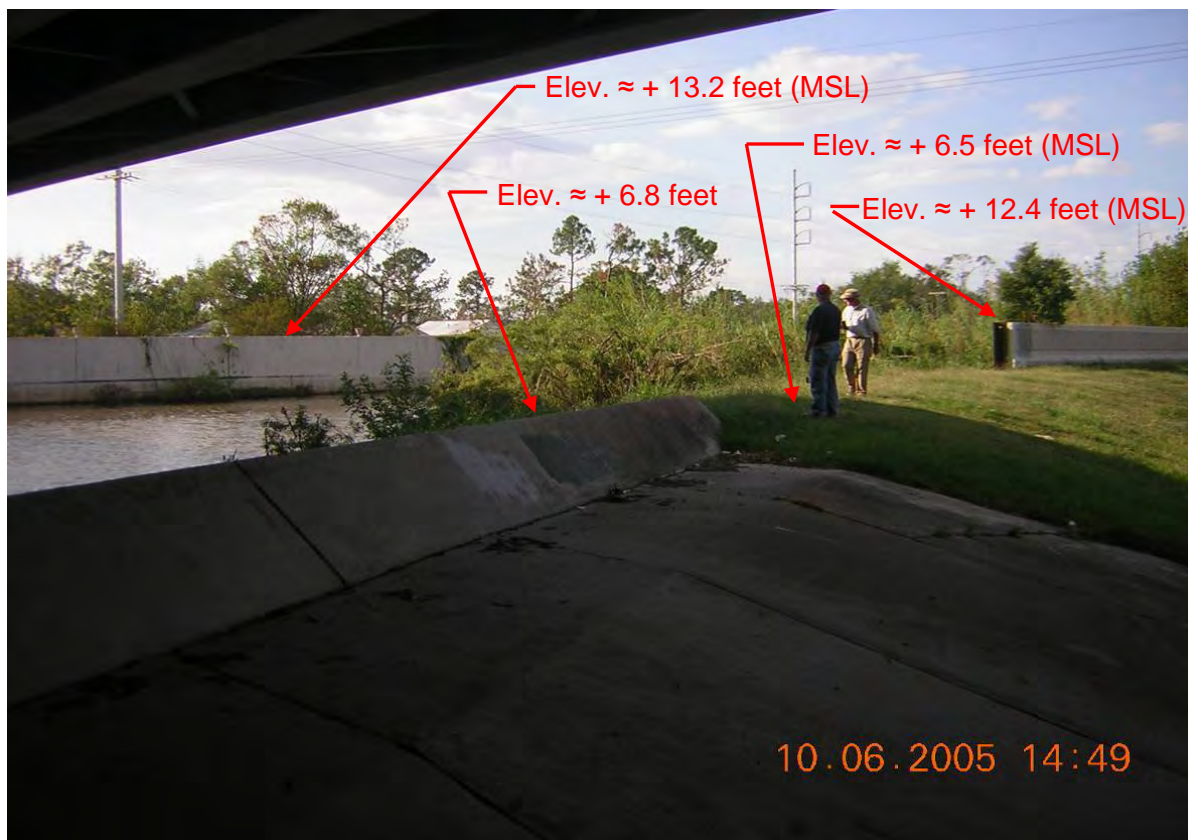
Source: IPET Report No. 2, April, 2006

Figure 8.18: Hydrograph proposed by IPET for the 17th Street drainage canal.



Photograph by Rune Storesund

Figure 8.19: View of the pump house, levees and floodwalls, and the "gap" at the south end of the Orleans canal.



Photograph by Rune Storesund

Figure 8.20: Side view of the "gap" at the south end of the Orleans canal.

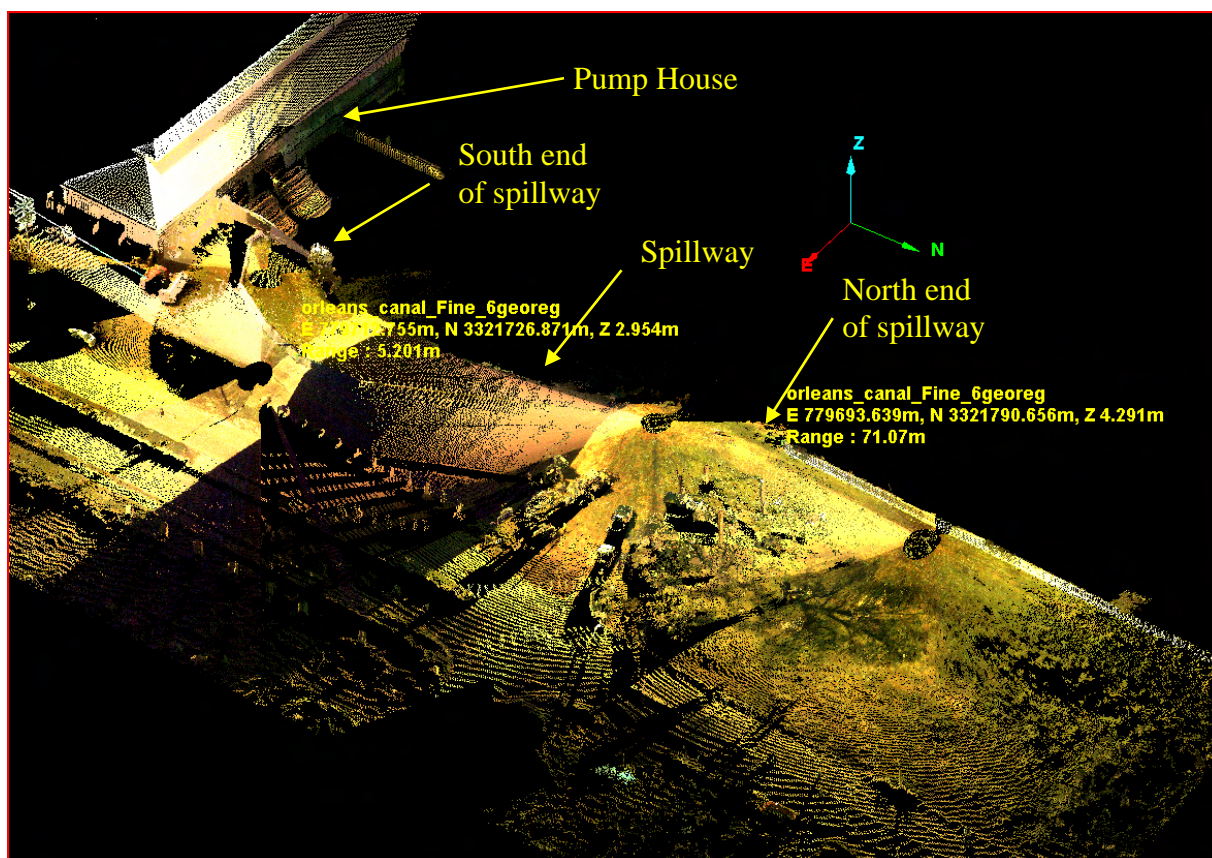


Figure 8.21: Oblique view of LIDAR survey of the southern end of the Orleans Canal, showing the “gap” on the east side where the levee and floodwall that should have sealed the end are omitted, and a “spillway” replaces them.



Photograph by Rune Storesund

Figure 8.22: View of the earthen levee embankments and floodwalls along the east side of the Orleans Canal (view looking to the North.)



Photograph by Rune Storesund

Figure 8.23: View of the earthen levee embankments and floodwalls along the west side of the Orleans Canal (view looking to the North.)

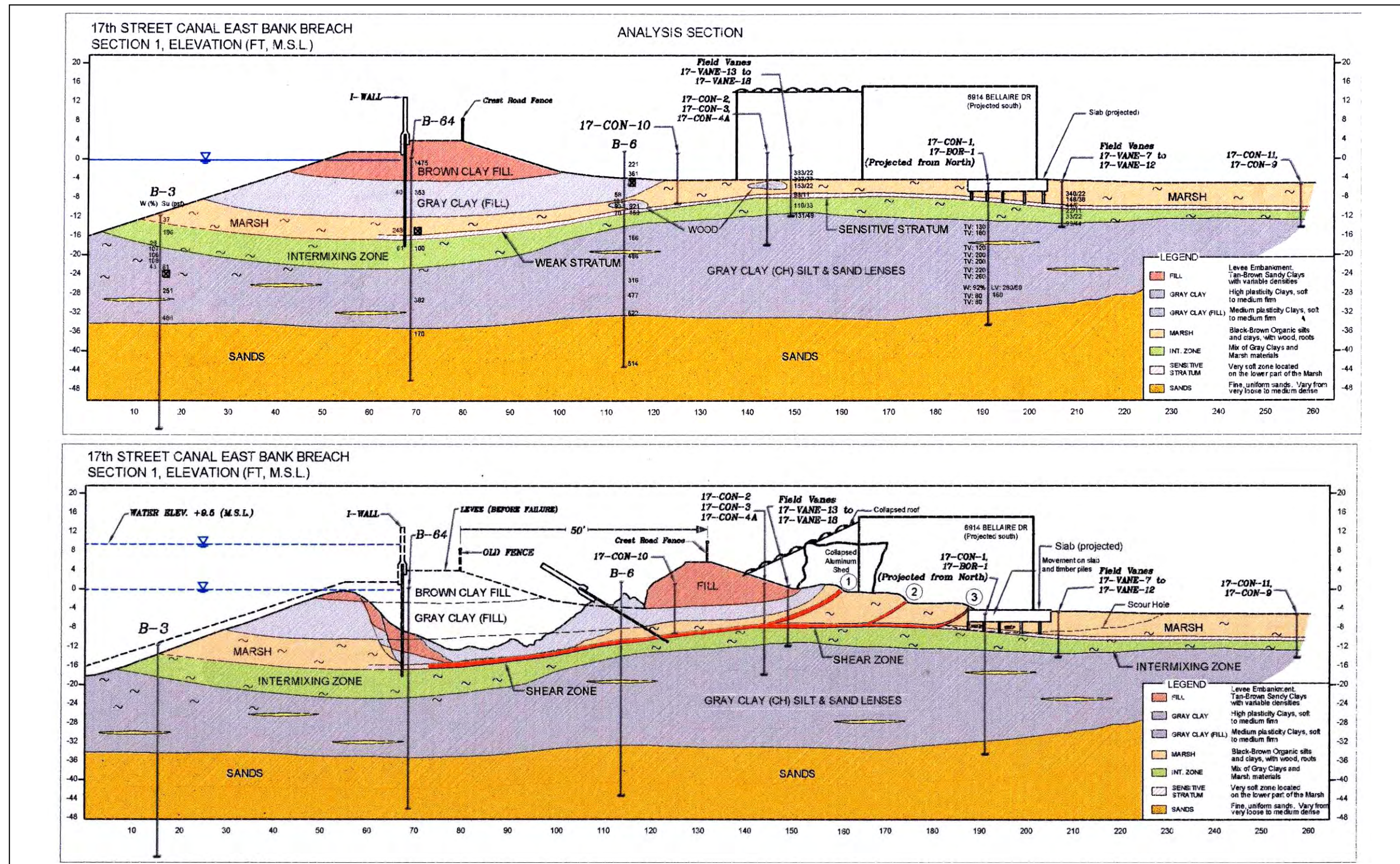


Figure 8.24: Oblique aerial view of the breach at the 17th Street Canal.





Figure 25(a): Plan view of 17th Street Canal showing locations of IPET borings and CPT



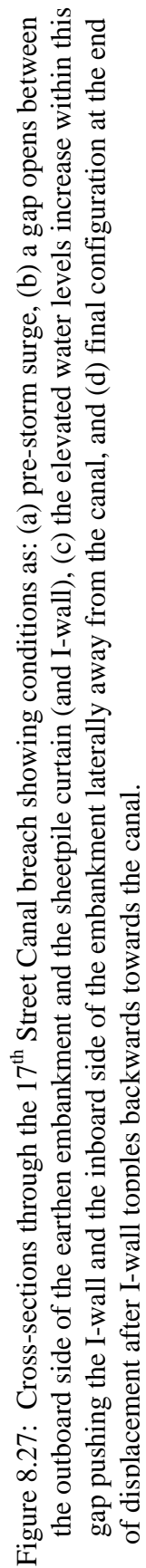




Figure 8.28: View of the 17th Street Canal breach site showing the tops of the I-wall sections that “pushed” (and followed) the displaced levee embankment section, and then toppled backwards towards the canal.



Figure 8.29: View of the 17th Street canal breach site showing the I-wall sections at the south end of the breach toppled forward (to the inboard side) by the intruding floodwaters. Note the severe scour inboard of the failure at this location.



Figure 8.30: Side view of the collapsed shed pushed into the house at 6914 Bellaire Drive as illustrated in the cross-section of Figure 8.27.



Photograph by Joseph Wartman

Figure 8.31: View of the foundation slab shown in the cross-section from Figure 8.27.



Figure 8.32: View of the exit of the upper failure surface (shear surface No.1 from Figure 8.27) at the inboard toe of the laterally translated embankment (toe scarp No. 1 in Figures 8.26 and 8.27).

[IPET Interim Report No. 2, April 1, 2006]



Figure 8.33: View of toe scarps No's. 2 and 3 at the inboard edge of the lateral translational failure at the 17th Street drainage canal.



Figure 8.34: Additional view of toe scarps No's. 2 and 3 at the inboard edge of the lateral translational failure at the 17th Street drainage canal.

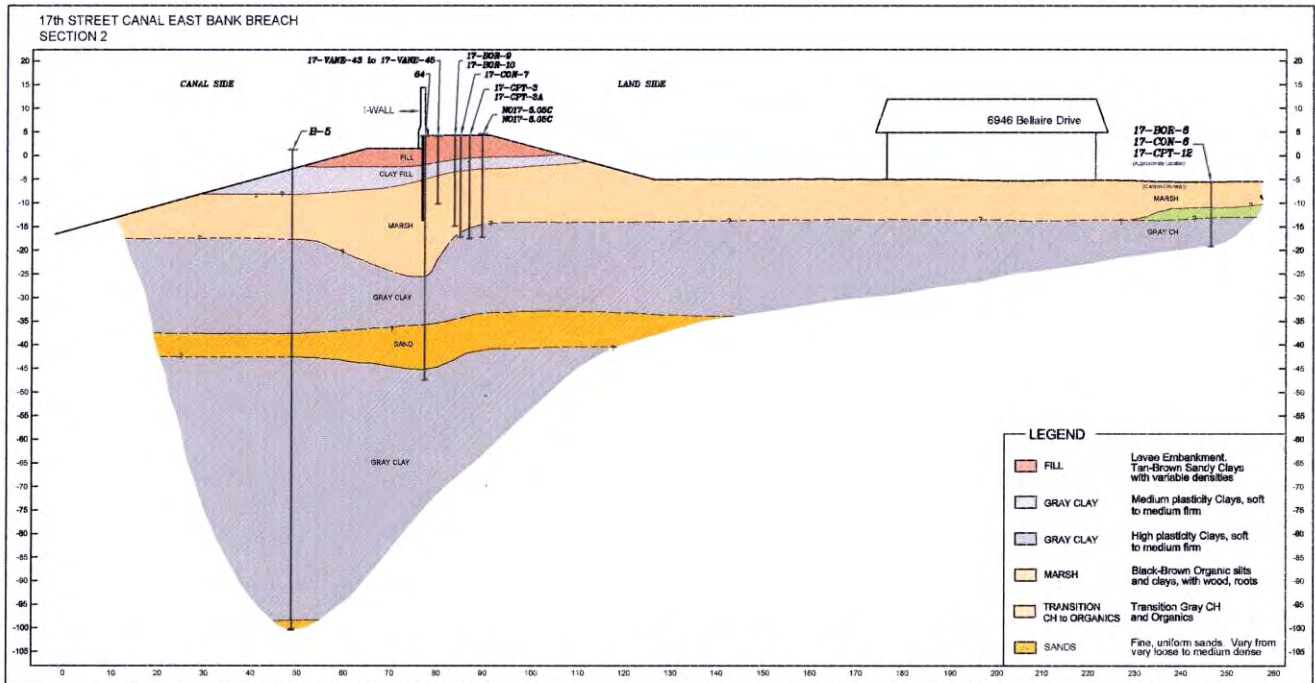
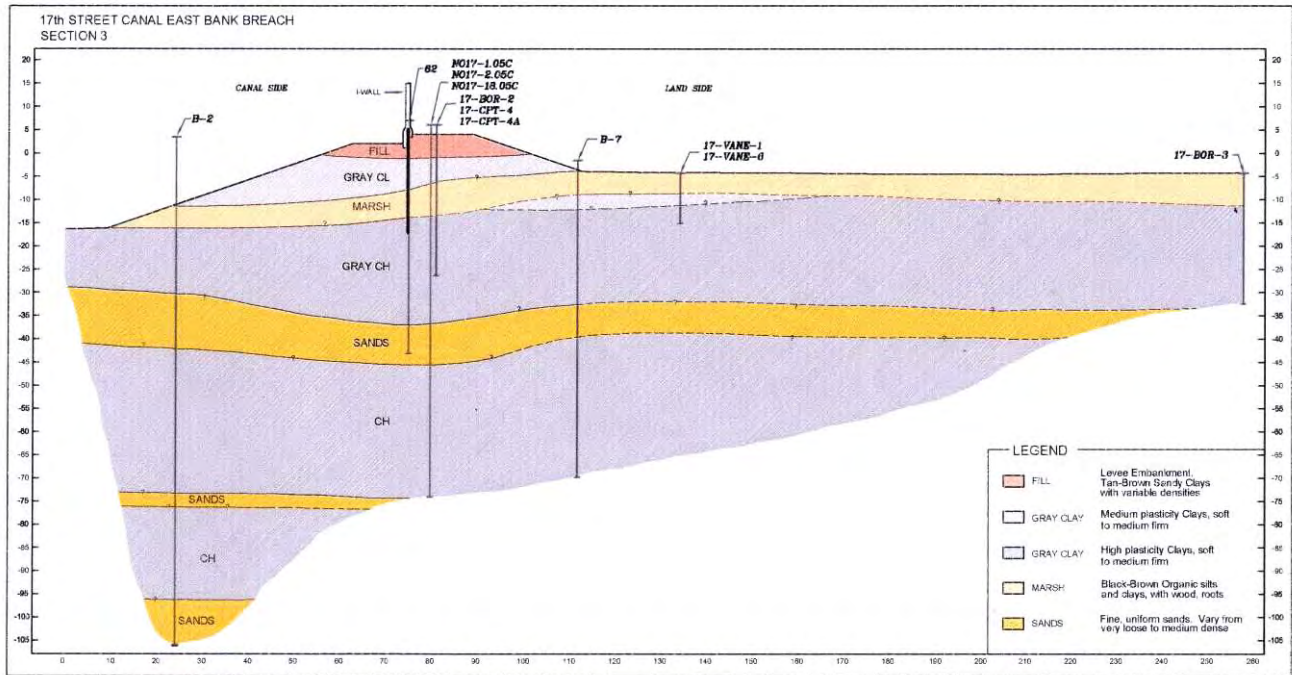


Figure 8.35: Two additional cross-sections through the breach section on the east bank of the 17th Street canal showing: (a) Cross-Section B-B' and (b) Cross-Section C-C'.

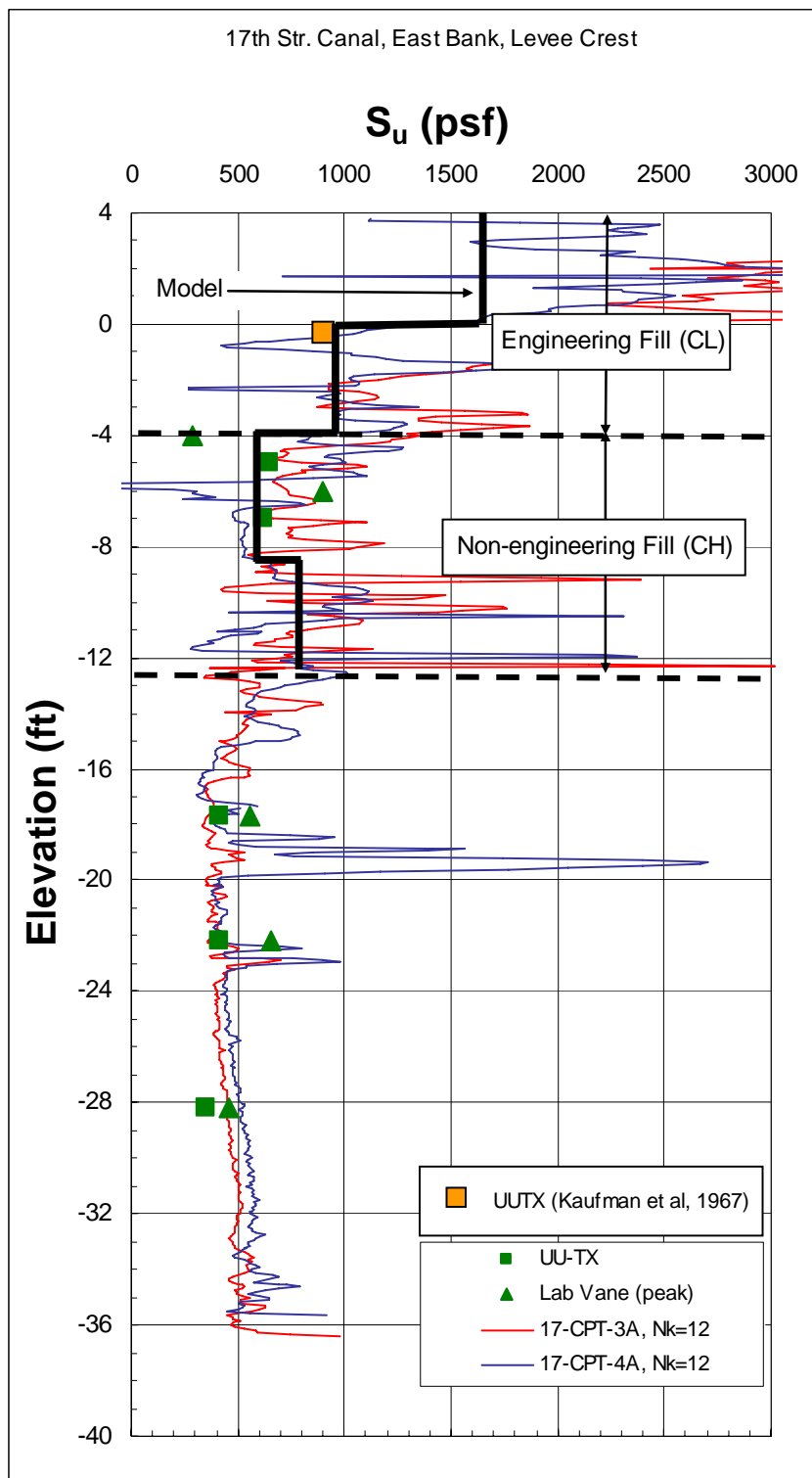
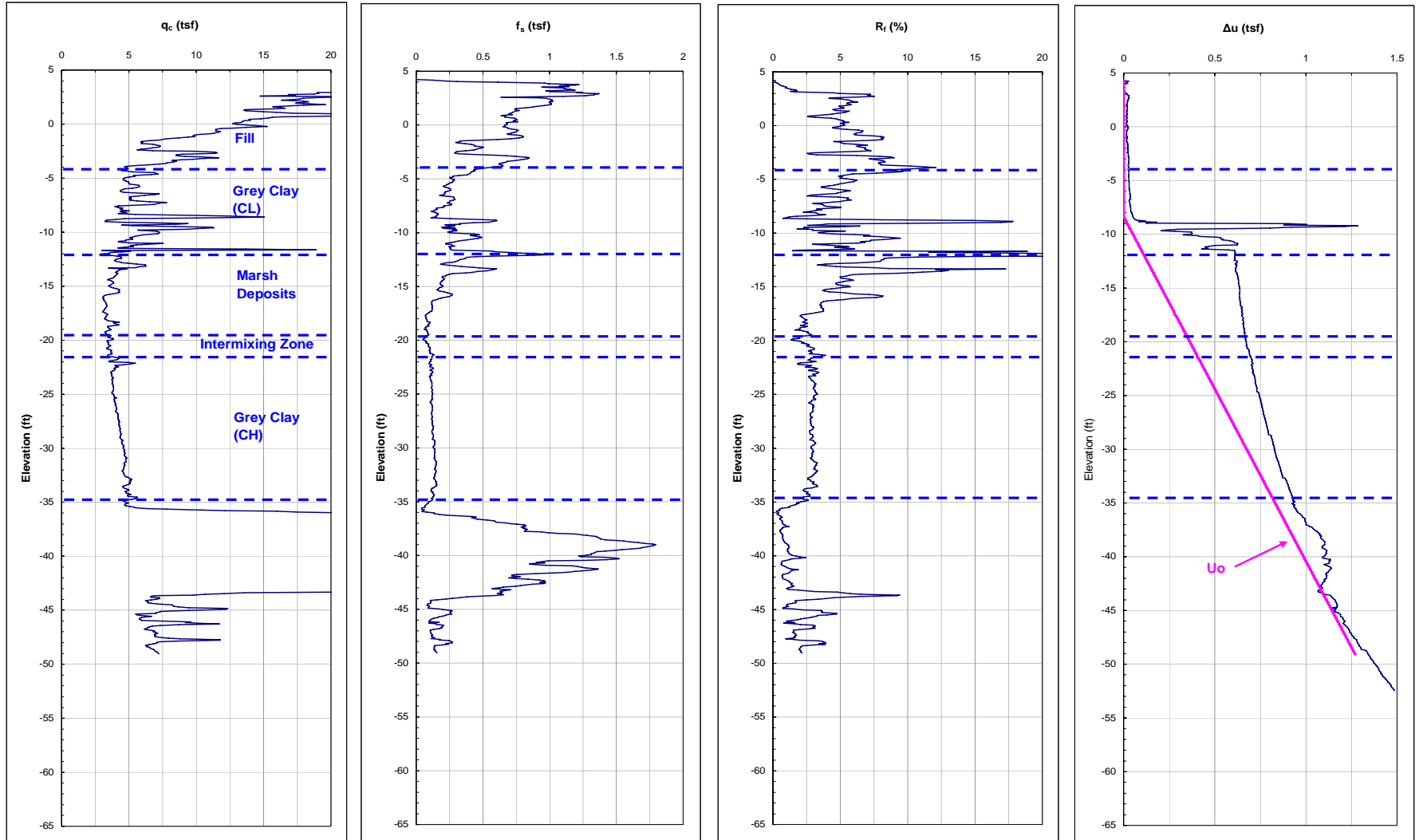


Figure 8.36: Summary of shear strength data within the 17th Street drainage canal levee embankment fill at and near the east bank breach section.



Example of CPT test data for locations beneath the levee embankment at and near the 17th Street canal breach section.

Figure 8.37:

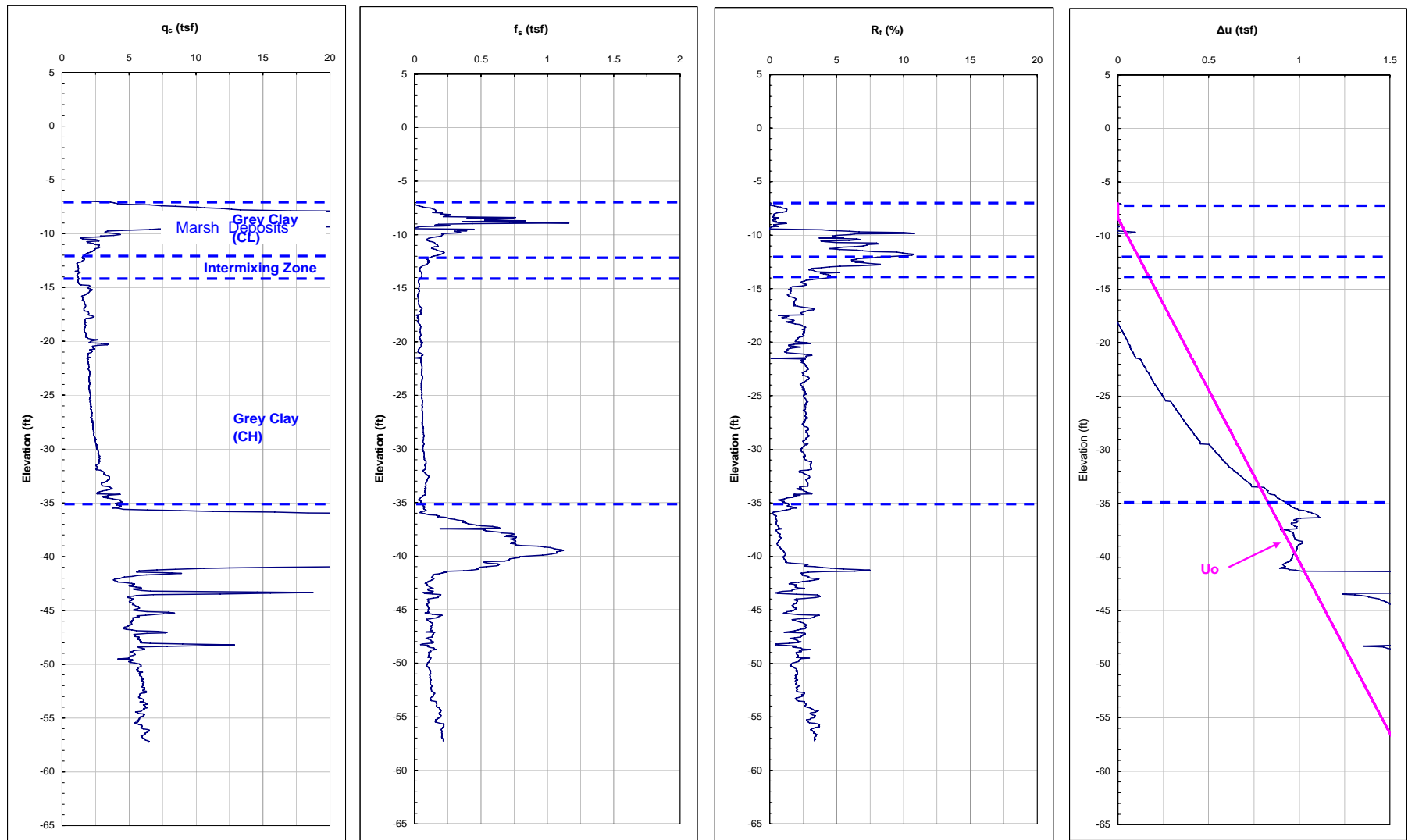


Figure 8.38: Example of CPT test data for locations inboard of the toe of the levee embankment at and near the 17th Street Canal breach section.

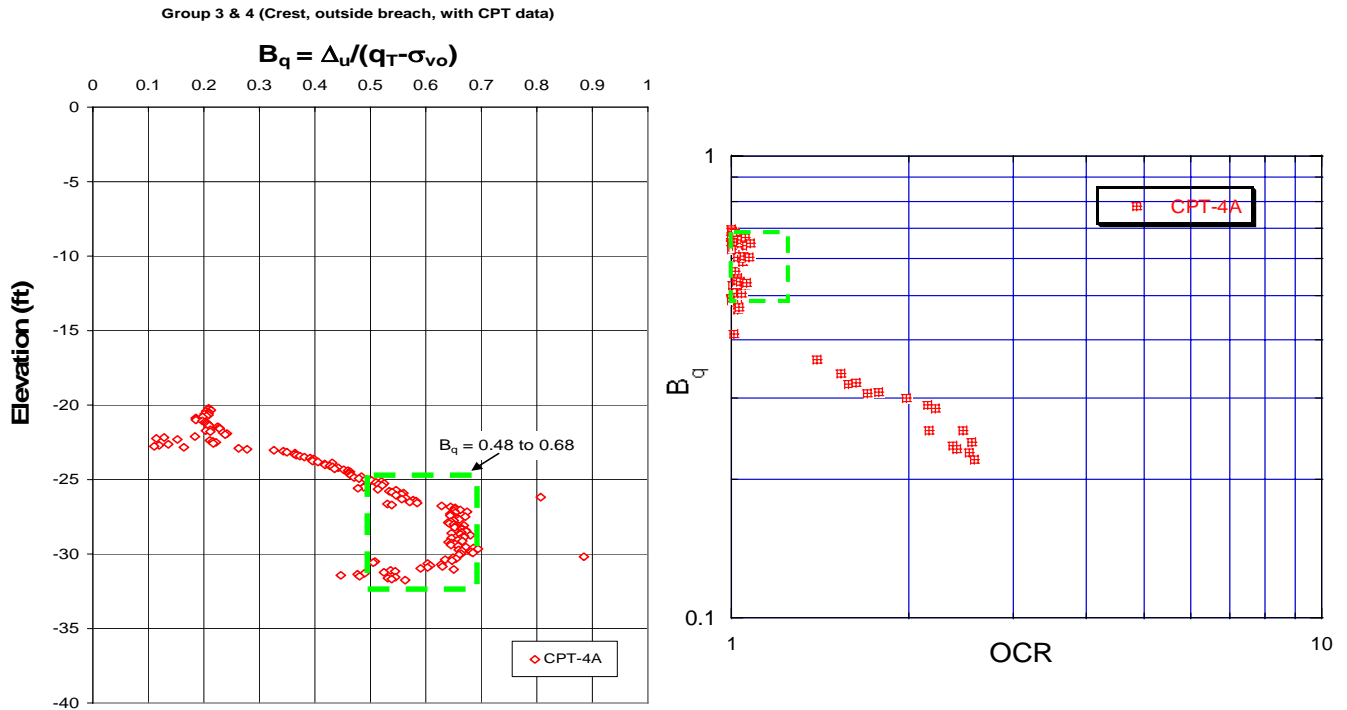


Figure 8.39: Pore Pressure parameter B_q vs. OCR and vs. depth within the soft gray clay (CH) foundation layer under full embankment overburden load.

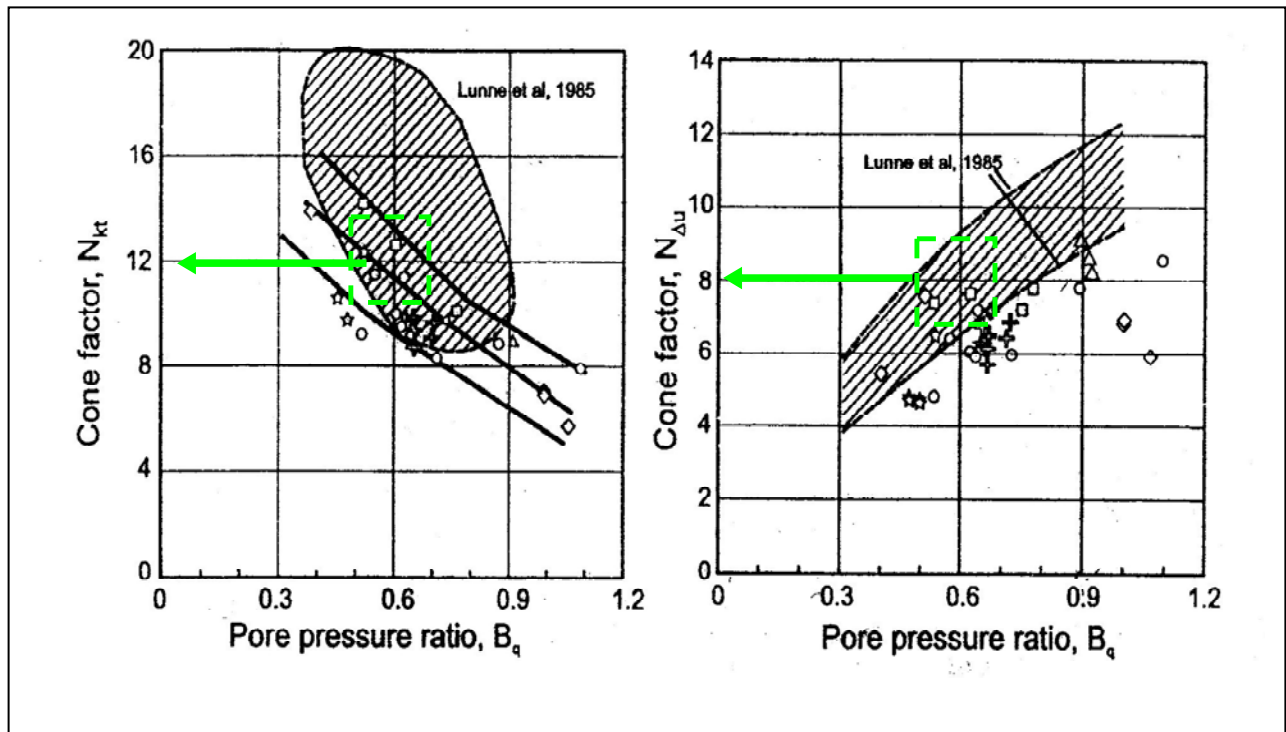


Figure 8.40: CPT cone factor N_{kt} based on B_q (after Lunne et al., 1985 and Karlsrud et al., 1996) for the soft gray clay (CH) under full embankment overburden load.

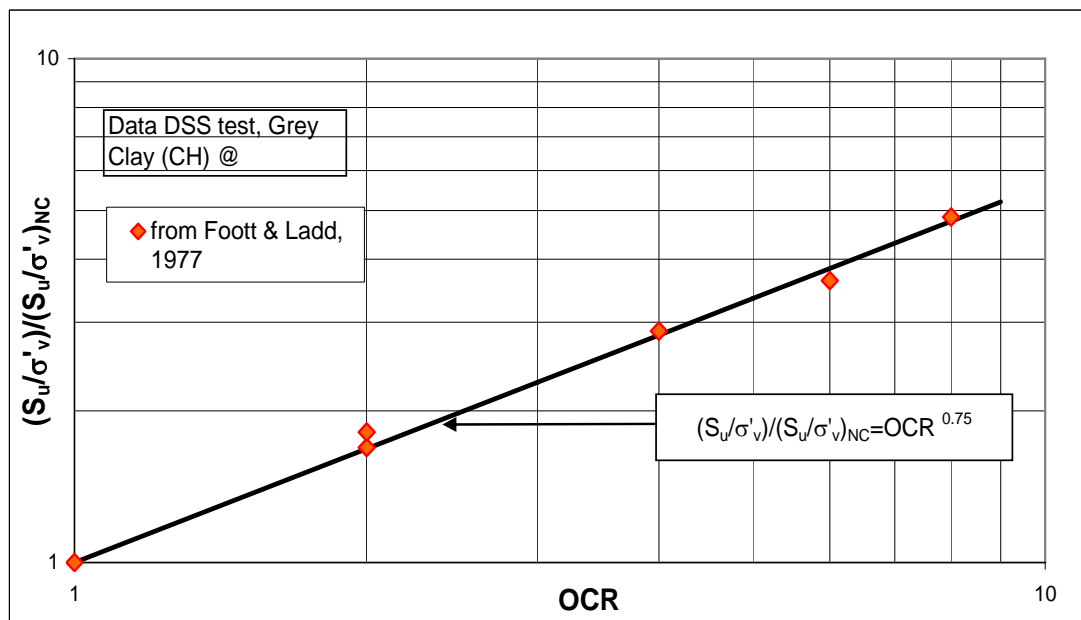


Figure 8.41: Relationship between S_u/P vs. OCR for mineralogically similar gray marsh clays (CH) at Atchafalía.

[Foott & Ladd, 1977]

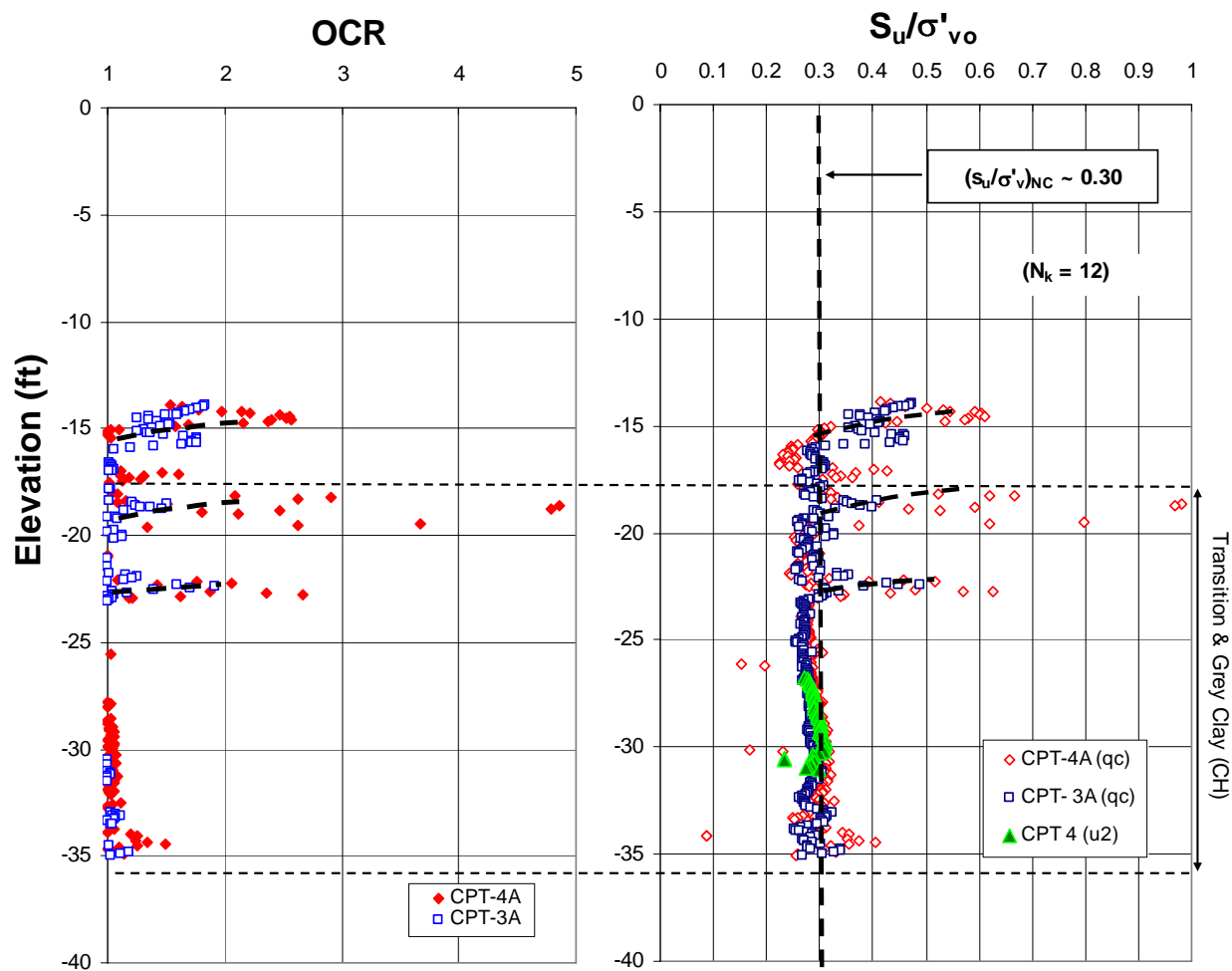


Figure 8.42: Plots of (a) OCR vs. Depth and (b) S_u vs. Depth for the soft gray marsh clay (CH) beneath the full embankment overburden pressure.

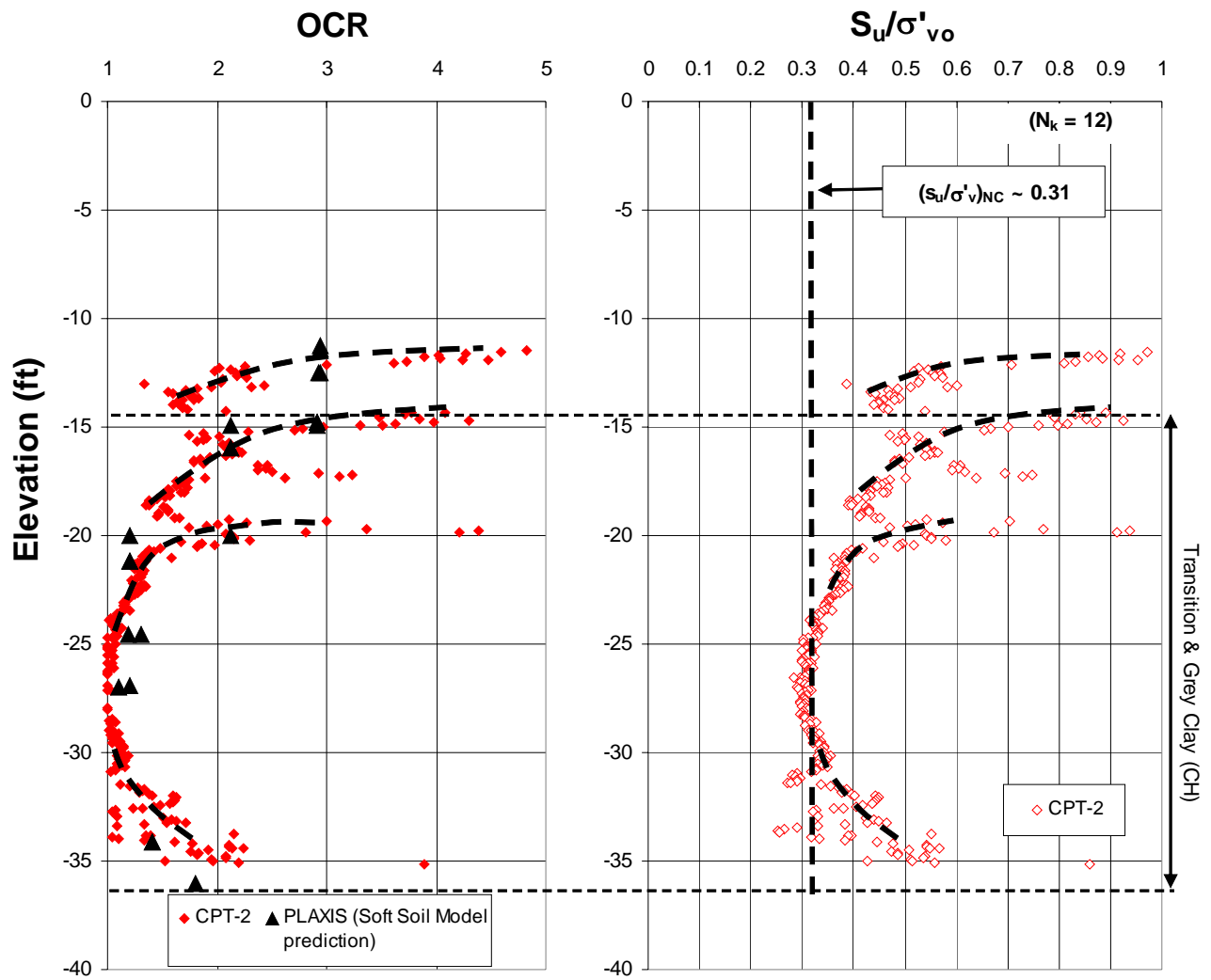
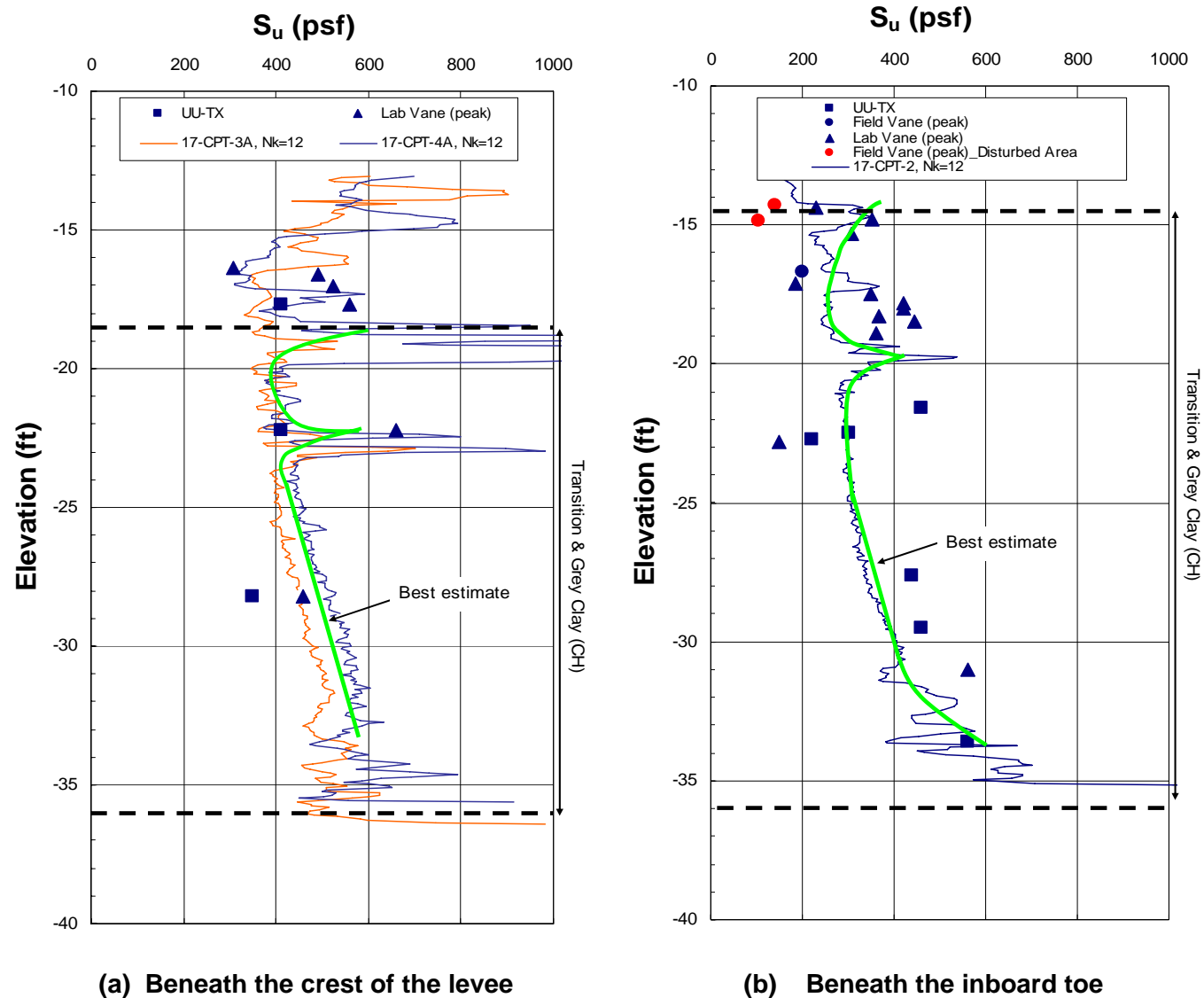


Figure 8.43: Plots of (a) OCR vs. Depth and (b) S_u vs. Depth for the soft gray marsh clay (CH) at the inboard toe and further to the landside (not under levee embankment overburden pressure) – 17th Street Canal breach site.



Profiles of shear strength vs. depth within the soft gray foundation clay at the 17th Street Canal breach site (a) beneath the crest of the levee, and (b) inboard of the toe of the levee.

Figure 8.44:

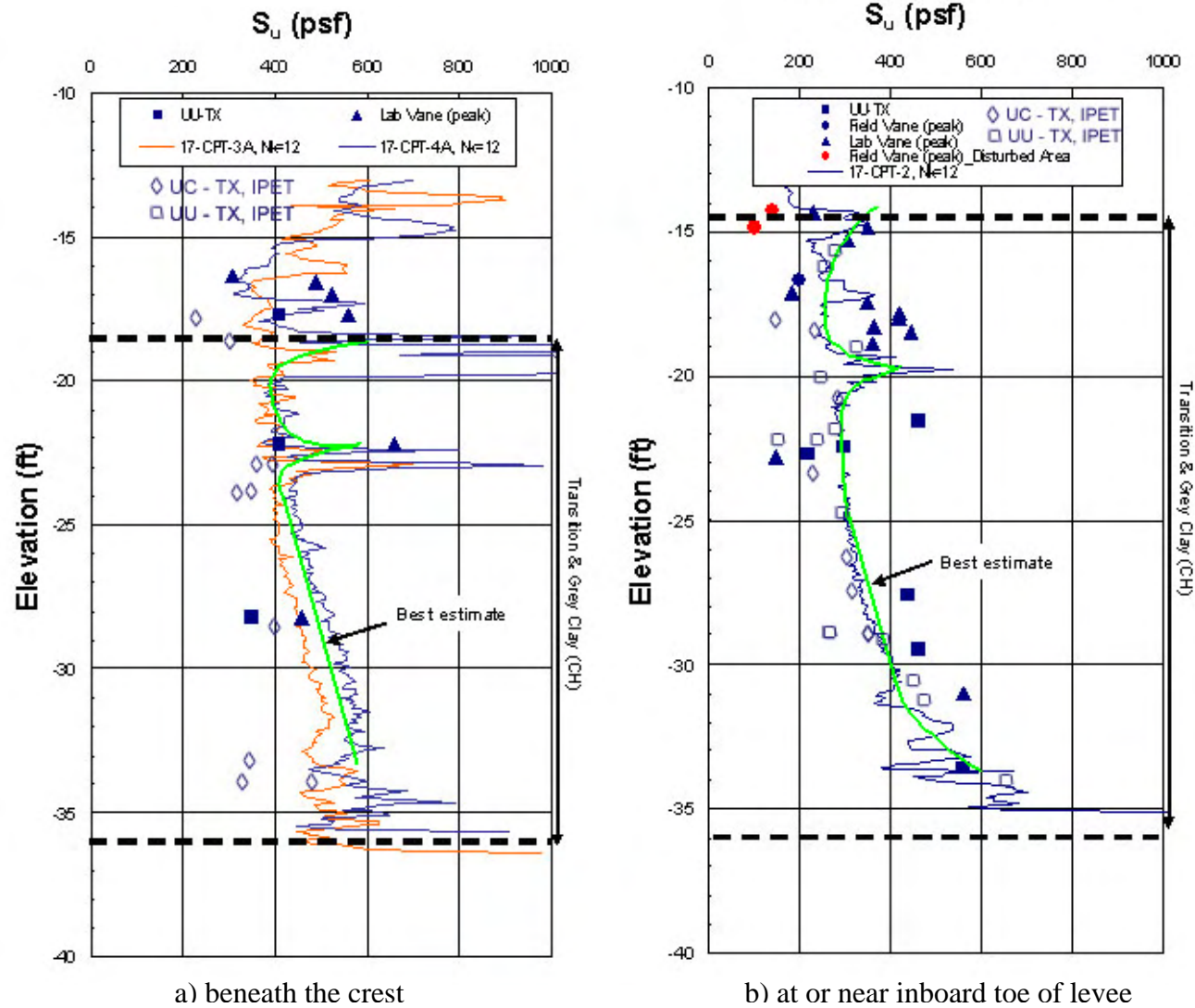


Figure 8.45: Profiles of shear strength vs. depth within the soft gray foundation clay at the 17th Street Canal breach site (a) beneath the crest of the levee, and (b) at and near the inboard toe of the levee, including the ILIT, IPET and pre-Katrina data.

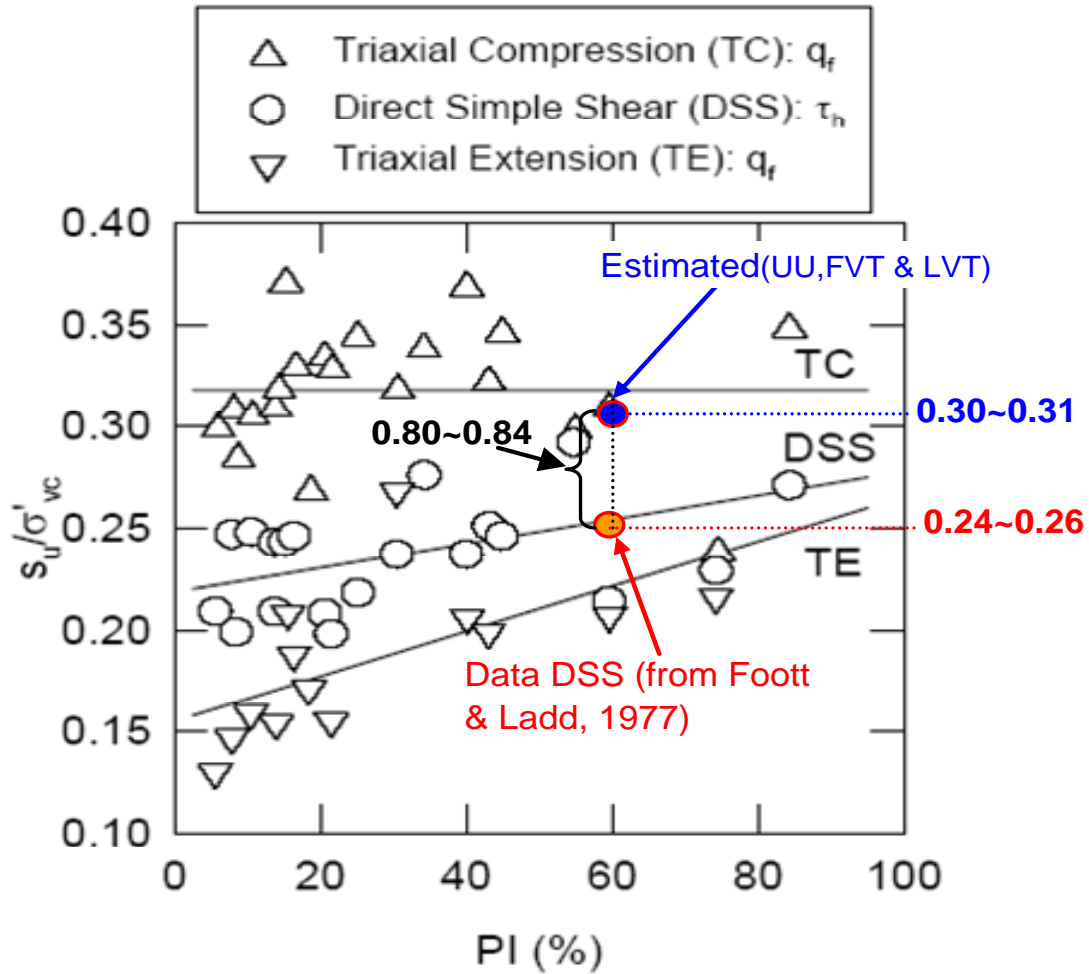


Figure 8.46: Undrained shear strength for UU-triaxial loading vs. undrained shear strength for DSS loading. [Base figure from Ladd, 2003]

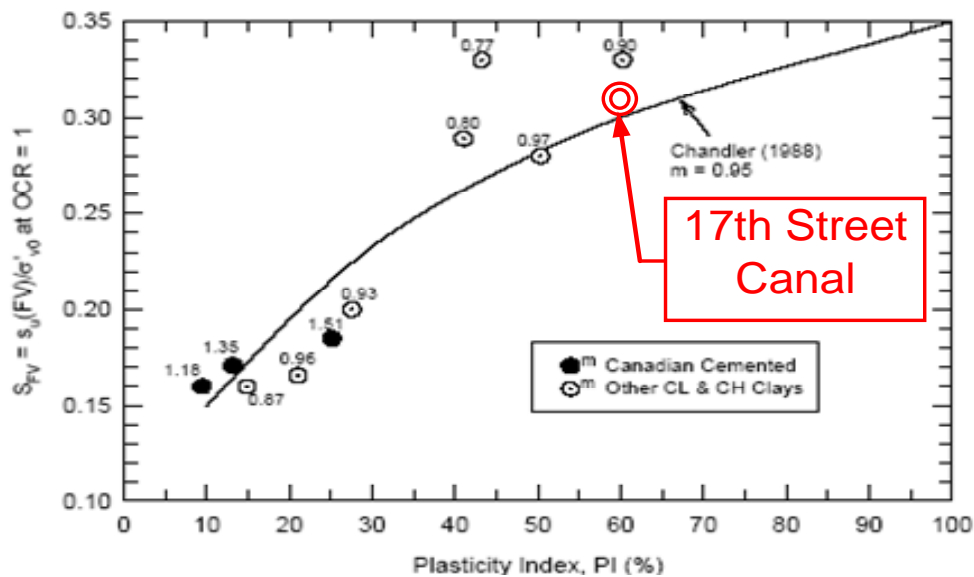


Figure 8.47: S_u/P for in situ field vane testing vs. Plasticity Index (PI) for normally consolidated clays (OCR = 1). [Base Figure from Ladd, 2003]

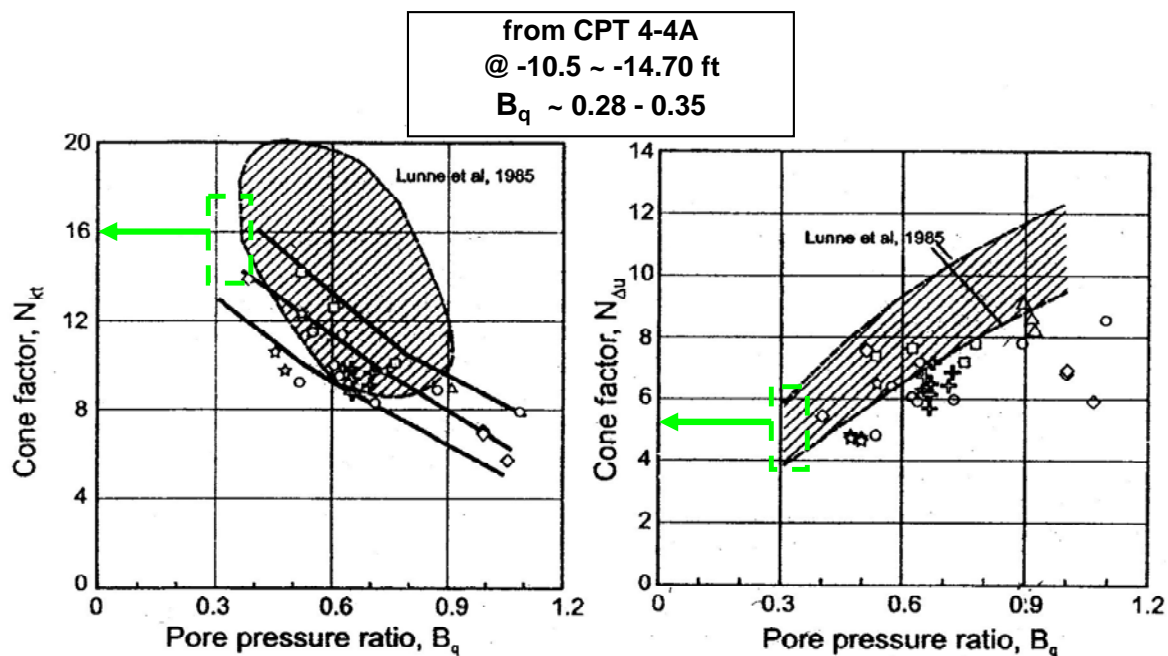


Figure 8.48: CPT cone factor N_{kt} based on B_q (after Lunne et al., 1985 and Karlsrud et al., 1996) for the marsh deposits under full embankment overburden load; 17th Street Canal breach site.

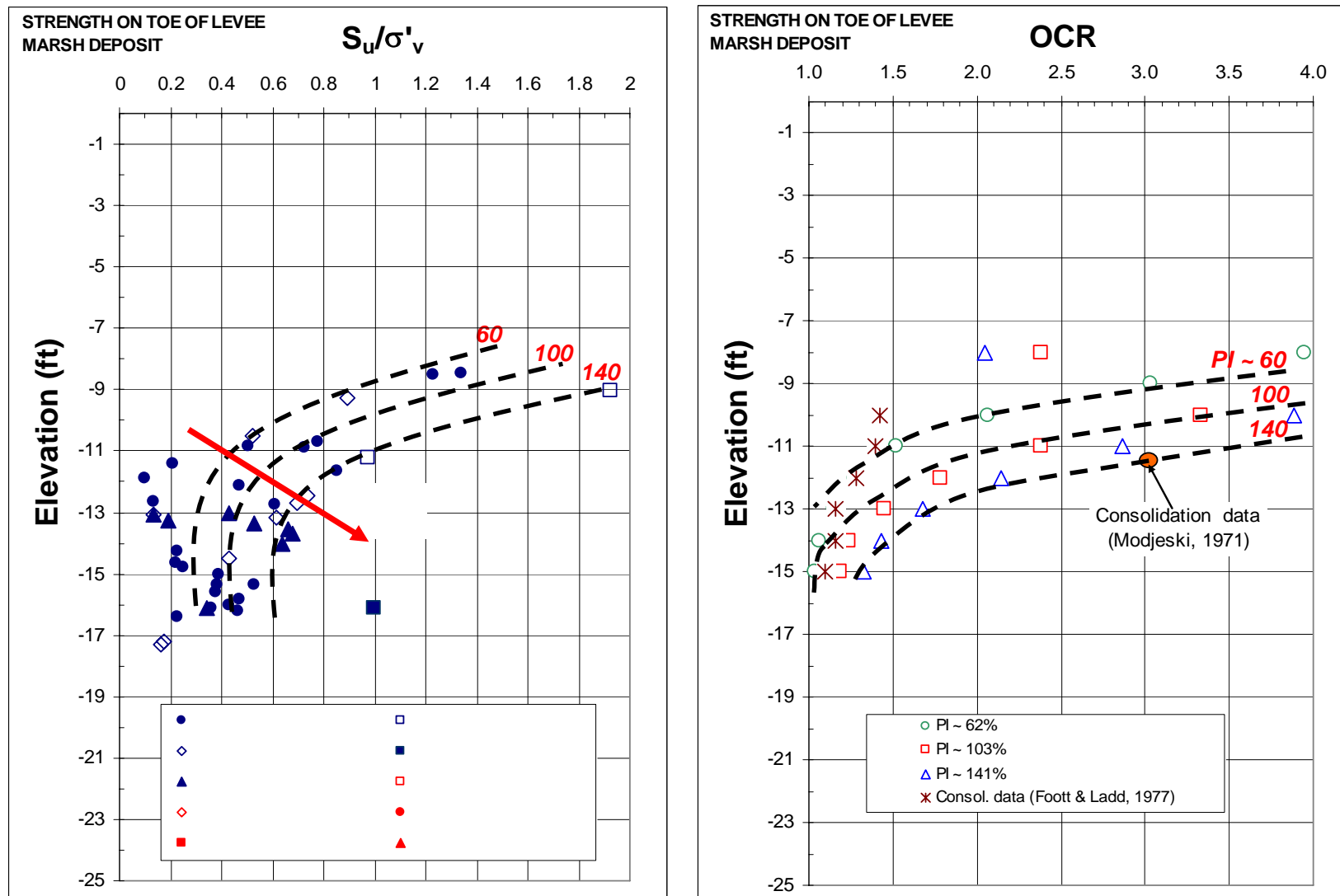
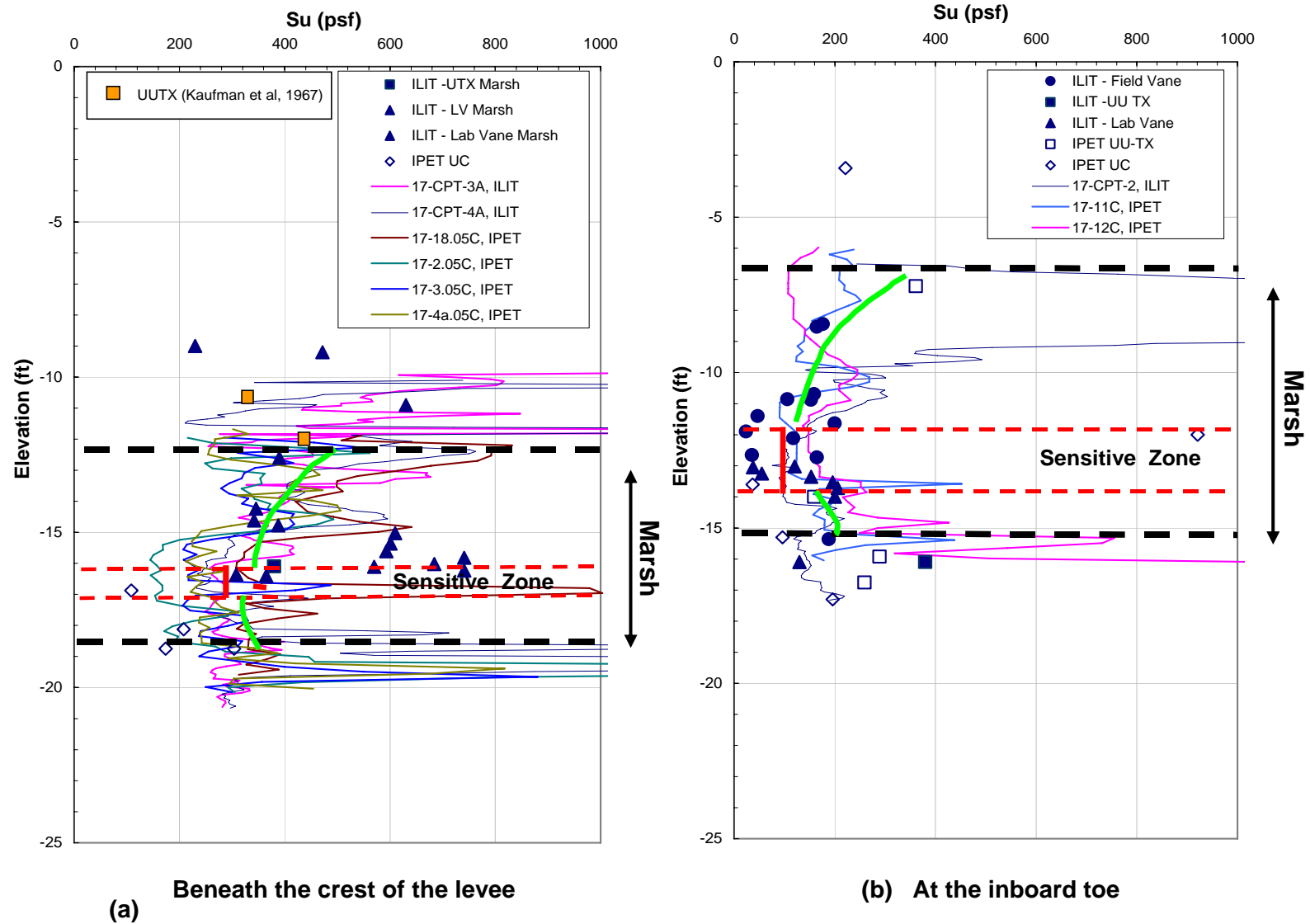


Figure 8.49: Undrained shear strength vs. depth, and OCR vs. depth, within the marsh deposits beneath the inboard toe based on th Street Canal breach site.

Mayne and Mitchell (1988) – 17



Shear strength vs. depth within the marsh deposits at the 17th Street Canal breach site (a) beneath the crest of the levee, and (b) at the inboard toe.

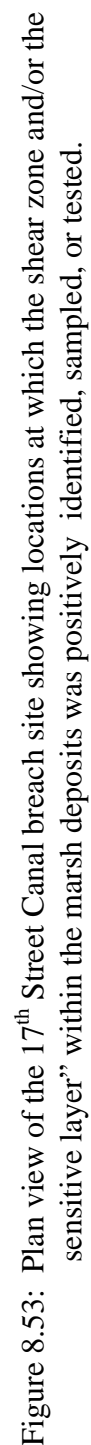
Figure 8.50:



Figure 8.51: Shiny dark brown to black sensitive organic clay on auger stem and (inset) closeup view of leaves and twigs; 17th Street Canal breach site.



Figure 8.52: Shear zone within disturbed area, middle of 17th Street Canal breach, adjacent to displaced block at Elev. -9 feet (MSL).



17th Str. Canal, West Bank



Figure 8.54: Photo of an “undisturbed” sample of the sensitive organic silty clay.



Figure 8.55: Laboratory vane shear testing of the thin layer of sensitive organic silty clay.

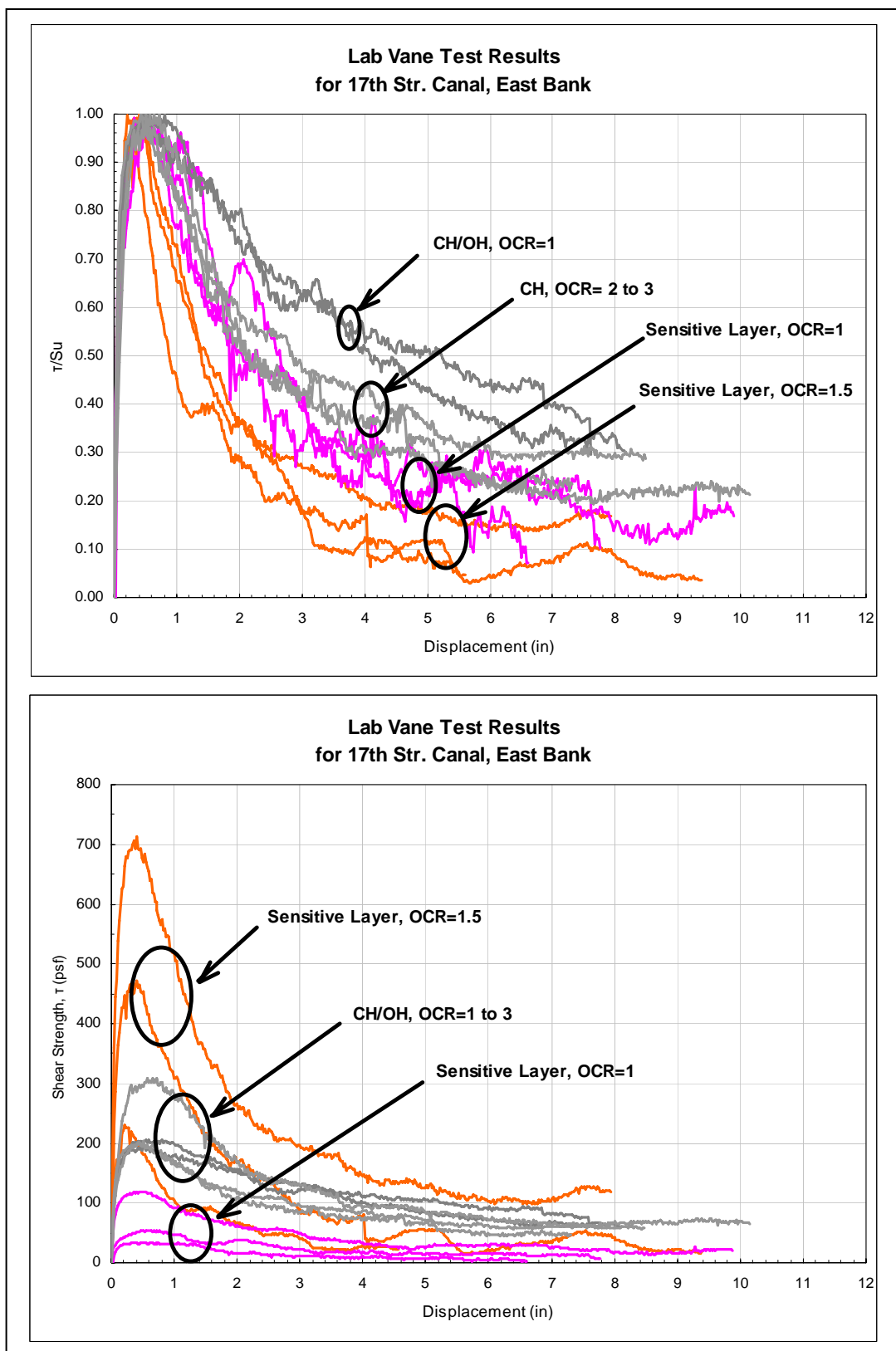


Figure 8.56: Stress-displacement behavior for soft gray clay (CH and CH/OH) and “sensitive” layer of organic clay within marsh deposit.

Geotechnical parameters for the Finite Element Analysis Model

ID	Soil Model	Name	Type	g_unsat [lb/ft ³]	g_sat [lb/ft ³]	k_x [ft/day]	k_y [ft/day]	nu [-]	E_ref [lb/ft ²]	c_ref [lb/ft ²]	phi [°]
CF Upper CH SC	Mohr Coulomb	ML-silt (Compacted Fill)	Undrained	105	115	0.0028	0.00028	0.35	234000	900	0.001
		CH-Upper Fat Clay	Undrained	90	95	0.00028	0.000028	0.35	180000	600	0.001
		SC (Sand)	Drained	110	110	0.28	0.28	0.30	1000000	0.01	38

ID	Soil Model	Name	Type	g_unsat [lb/ft ³]	g_sat [lb/ft ³]	k_x [ft/day]	k_y [ft/day]	lambda* [-]	kappa* [-]	n_ur [-]	K0nc [-]	M [-]	c_ref [lb/ft ²]	phi @ pref [°]	OCR
M2 M1 IZ CH1 CH2-a CH2-b	Soft Soil	Marsh 2	Undrained	80	80	28.3	2.8	0.21	0.033	0.15	0.60	1.90	0	36	2.25
		Marsh 1	Undrained	80	80	28.3	2.8	0.21	0.033	0.15	0.60	1.90	0	36	1.1
		Intermixing zone	Undrained	85	85	28.32	2.8	0.10	0.02	0.15	0.61	1.27	0	23	3
		Grey Clay	Undrained	90	95	0.00028	0.000028	0.17	0.03	0.15	0.63	1.24	0	22	2.2
		Grey Clay	Undrained	90	95	0.00028	0.000028	0.17	0.03	0.15	0.63	1.24	0	22	1.25
		Grey Clay	Undrained	90	95	0.00028	0.000028	0.17	0.03	0.15	0.63	1.24	0	22	1.5

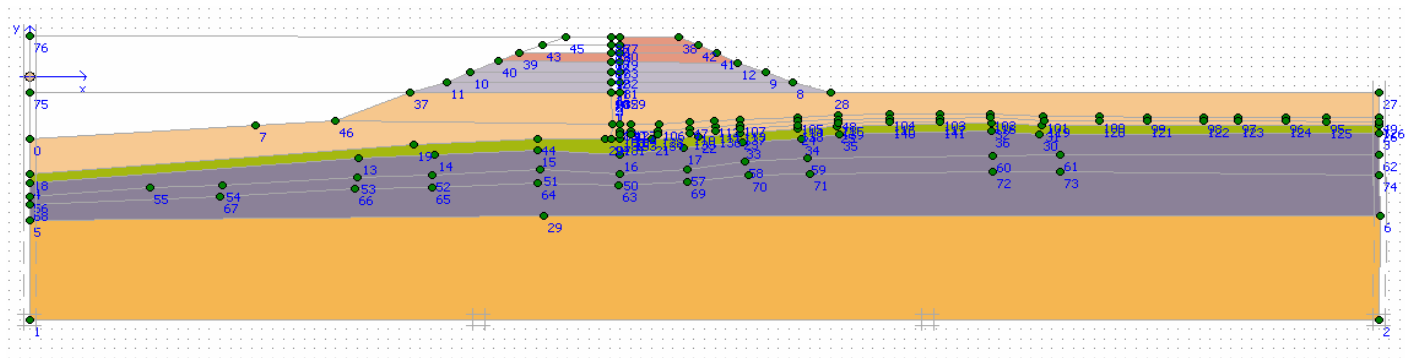


Figure 8.57: Parameters (and model) used in PLAXIS model for the 17th Street Canal breach site.

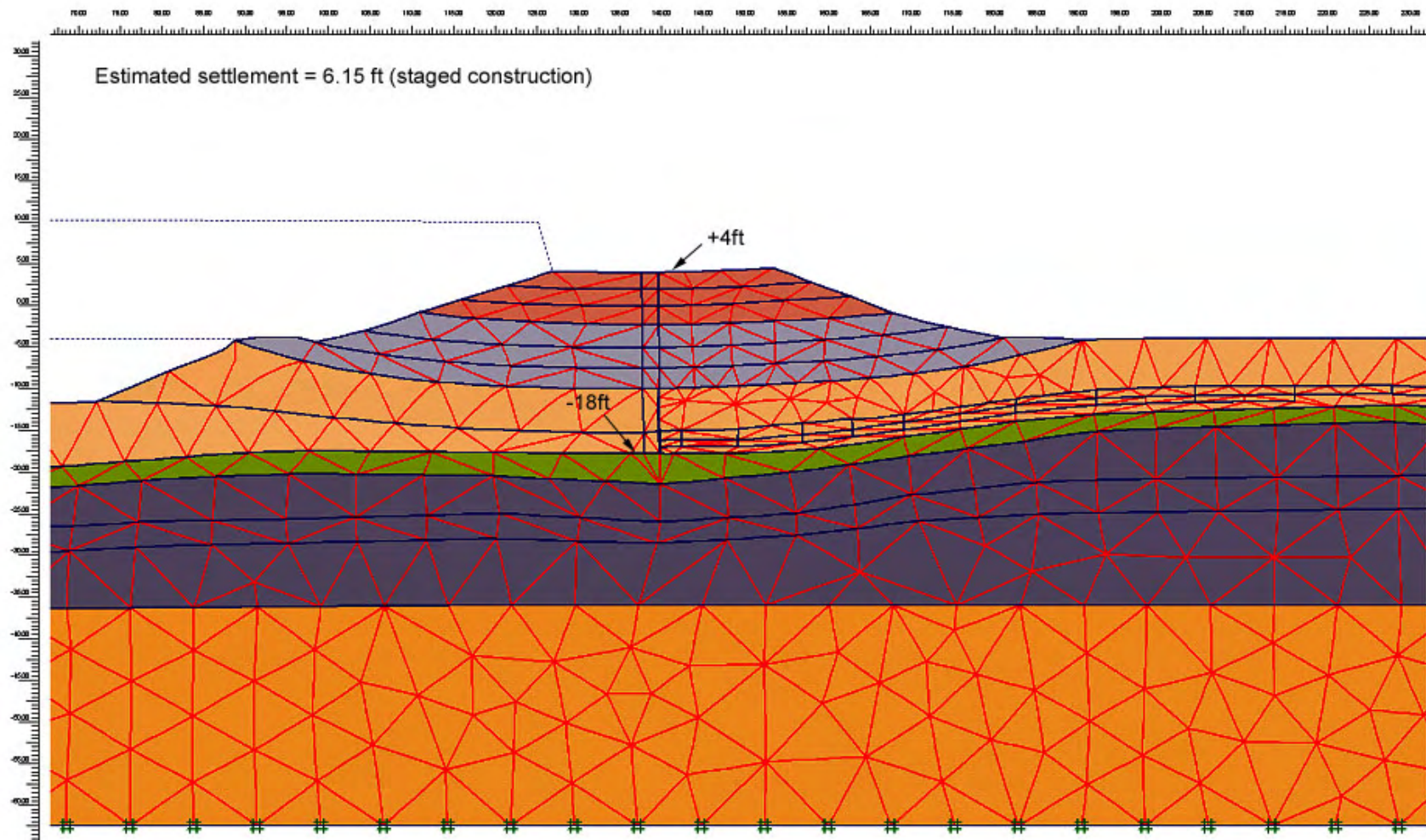
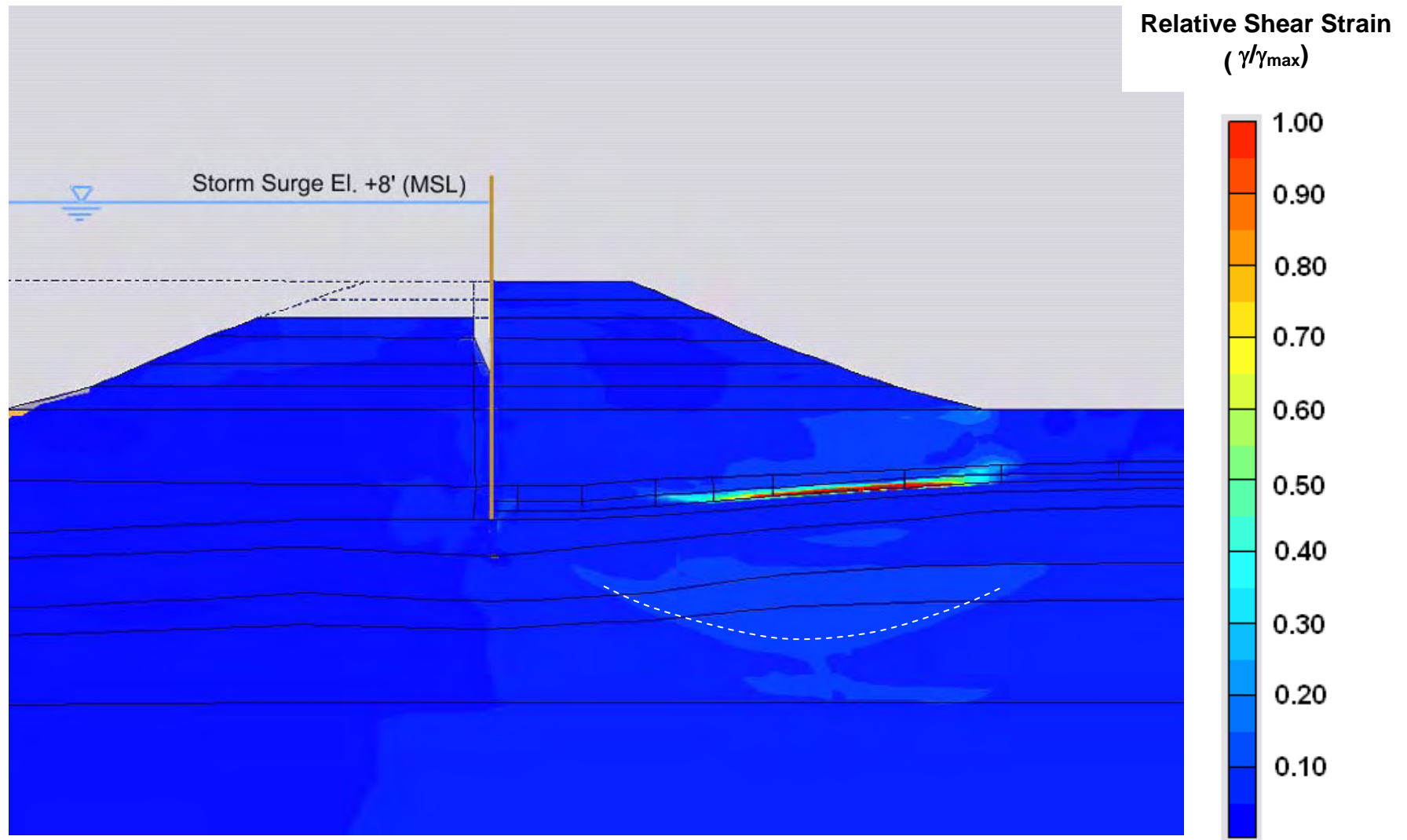
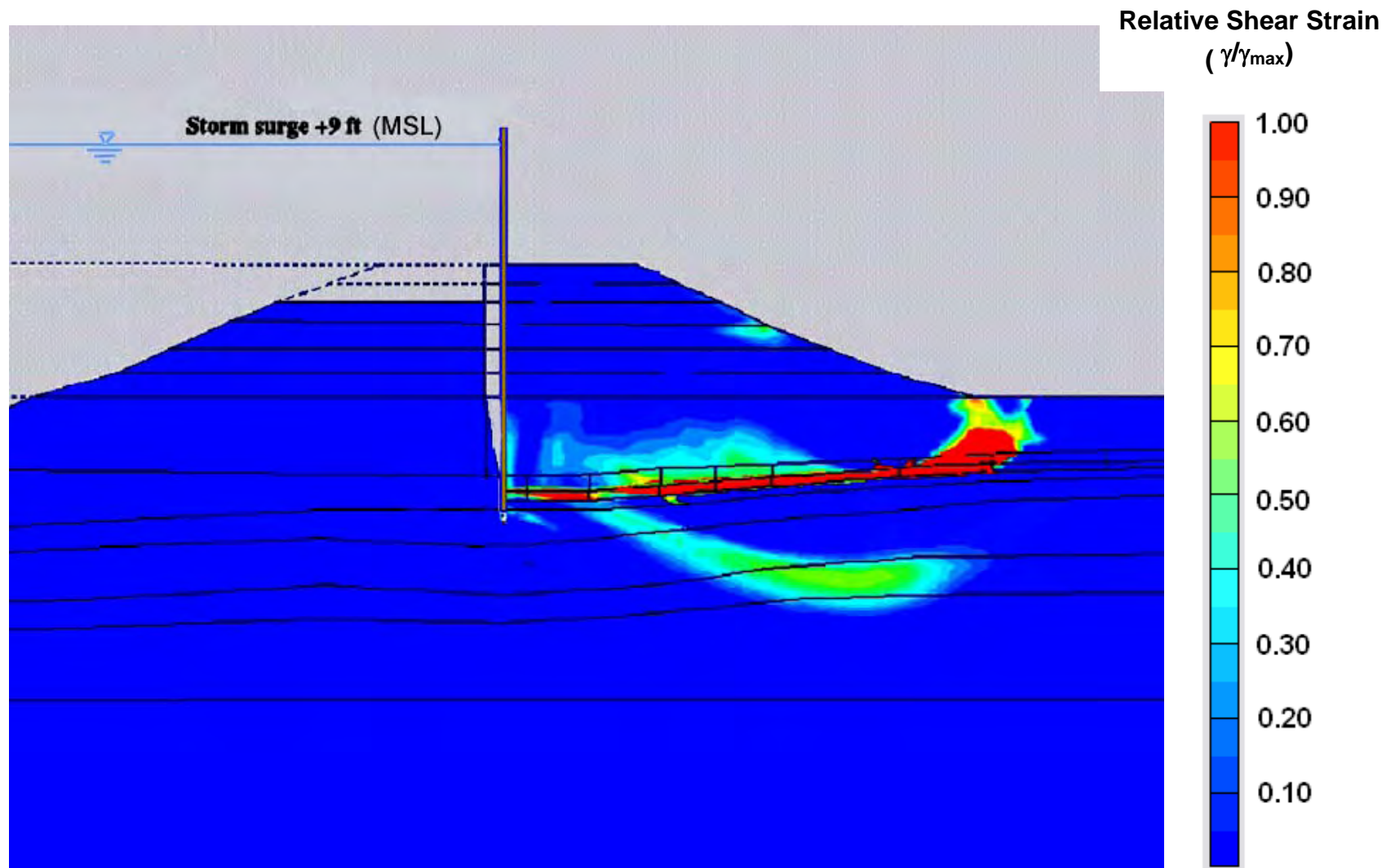


Figure 8.58: Deformed mesh at the end of initial construction of embankment and consolidation; 17th Street Canal breach section.



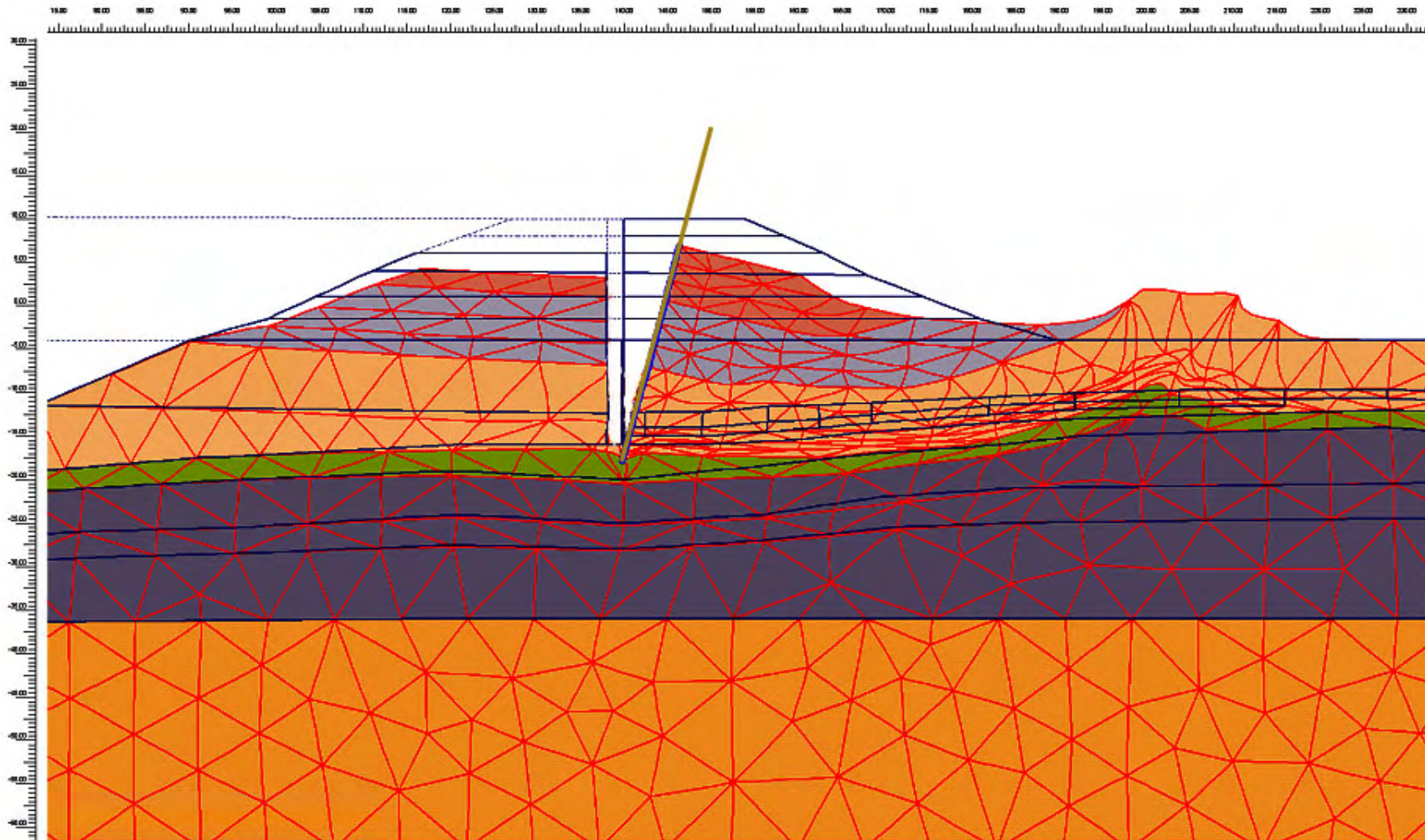
Normalized shear strain contours (shear strain divided by strain to failure) for a storm surge at Elev. + 8 feet (MSL)
At the 17th Street Canal breach site; initiation of gapping at outboard toe of floodwall.

Figure 8.59:



Normalized shear strain contours (shear strain divided by strain to failure) for a storm surge at Elev. + 8 feet (MSL) at the 17th Street Canal breach site; gapping at outboard toe of floodwall is now developed to full depth.

Figure 8.60:



Displaced mesh for storm surge height at Elev. + 8.5 feet (MSL) at the 17th Street Canal breach site; displacements are
Figure 8.61: increased by a factor of 3 in this figure.

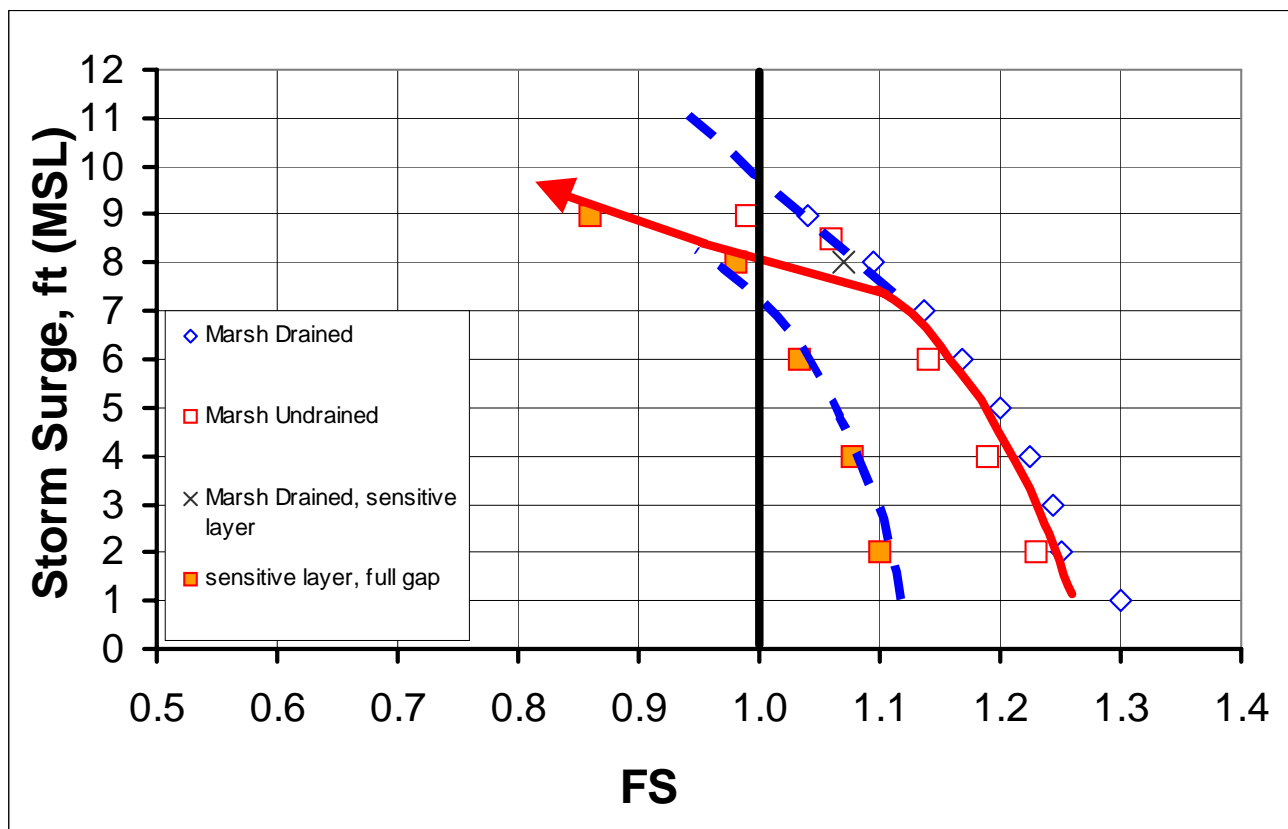


Figure 8.62: Calculated Factors of Safety for three modes based on PLAXIS analyses of the 7th Street Canal breach section for various canal water elevations; showing the best-estimated path to failure.

ID	Soil Model	Type	γ	c	ϕ	Su/p'
			[lb/ft ³]	[lb/ft ²]	[°]	
Upper fill	Mohr-Coulomb	Undrained	110	800	0	
Lower fill	Mohr-Coulomb	Undrained	85	550	0	
Marsh, beneath crest	Mohr-Coulomb	Undrained	80	375	0	
Marsh, free field	Mohr-Coulomb	Undrained	80	200	0	
Sensitive layer, beneath crest	Mohr-Coulomb	Undrained	80	240	0	
Sensitive layer, beneath toe	Mohr-Coulomb	Undrained	80	180	0	
Sensitive layer, free field	Mohr-Coulomb	Undrained	80	70	0	
Overconsolidated Grey CH	Mohr-Coulomb	Undrained	90	400	0	
Normally consolidated Grey CH	SHANSEP	Undrained	90			0.26
Sand	Mohr-Coulomb	Drained	110	0	33	

Geotechnical Parameters for the Limit Equilibrium Analyses for 17th Street Canal levee, East Bank.

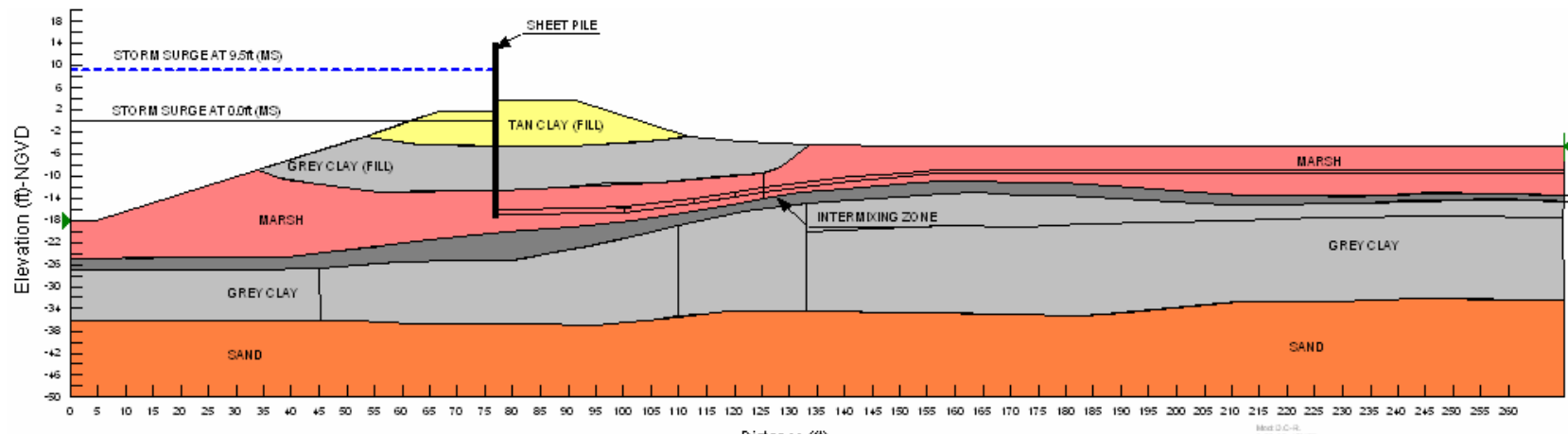


Figure 8.63: Cross-section and parameters used for conventional stability analyses of the breach section on the east side of the 17th Street Canal.

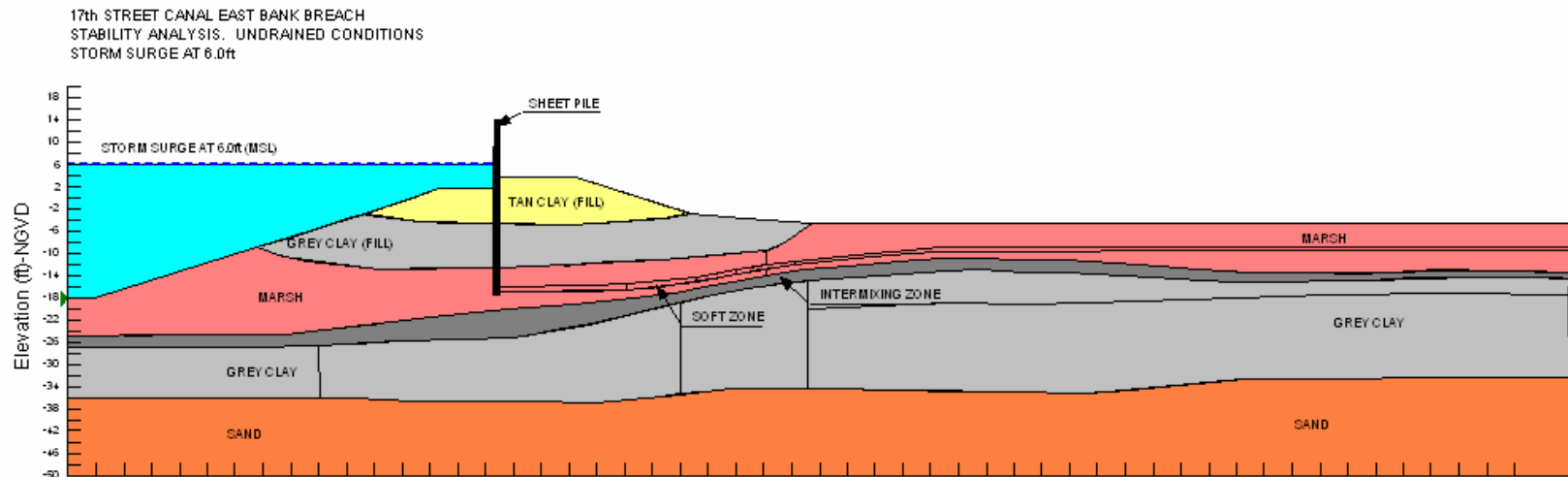


Figure 8.64: Cross-section at 17th Street Canal breach site for static slope stability analyses.

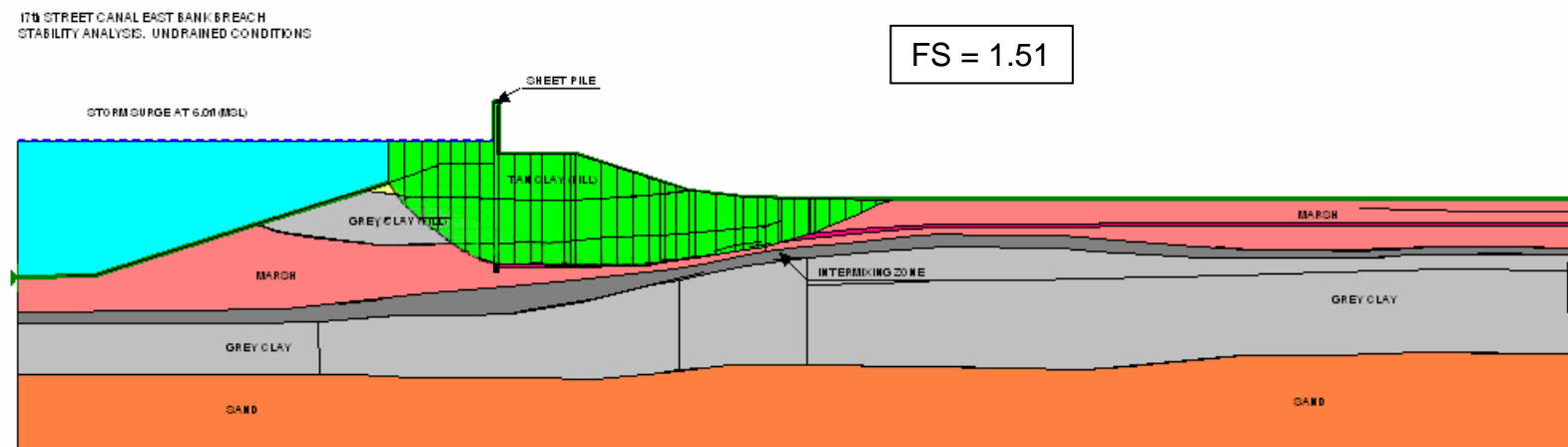


Figure 8.65: Stability analysis (worst case) for storm surge to Elev. + 6 feet (MSL) at the 17th Street Canal breach section.

17th STREET CANAL EAST BANK BREACH
STABILITY ANALYSIS. UNDRAINED CONDITIONS

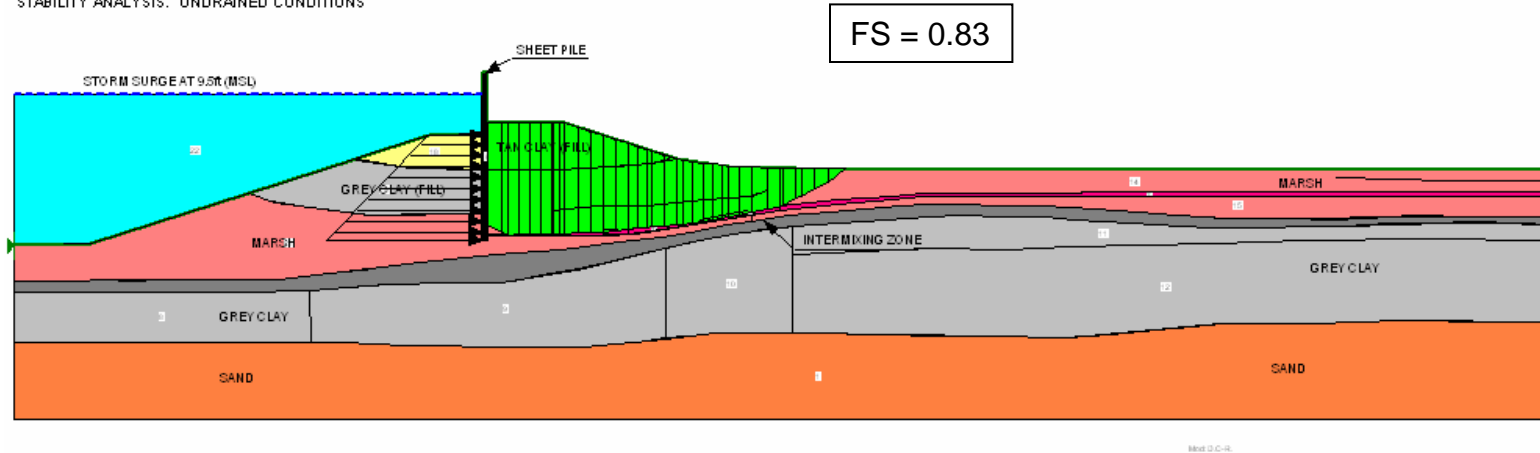


Figure 8.66: Stability analysis for actual observed failure mechanism at the 17th Street Canal breach section, with storm surge at Elev. + 9.5 feet (MSL) and with fully developed crack at the outboard side of the sheetpile/floodwall.

17th STREET CANAL EAST BANK BREACH
STABILITY ANALYSIS. UNDRAINED CONDITIONS

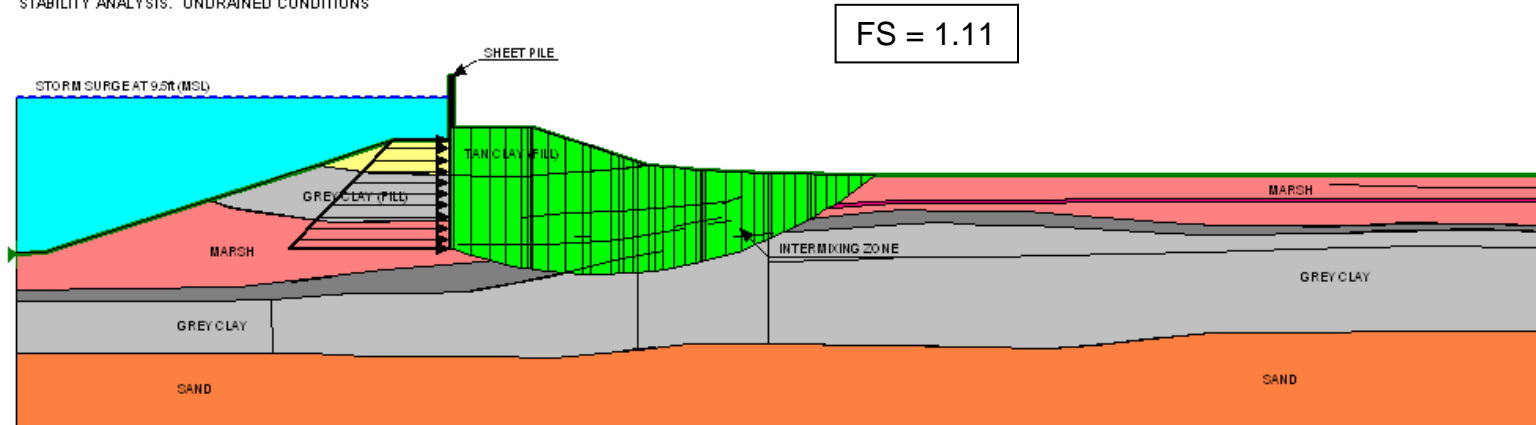


Figure 8.67: Stability analysis for shear failure through the deeper soft gray clays (CH) at the 17th Street Canal breach section, with storm surge at Elev. + 9.5 feet (MSL) and with fully developed crack at the outboard side of the sheetpile/floodwall.

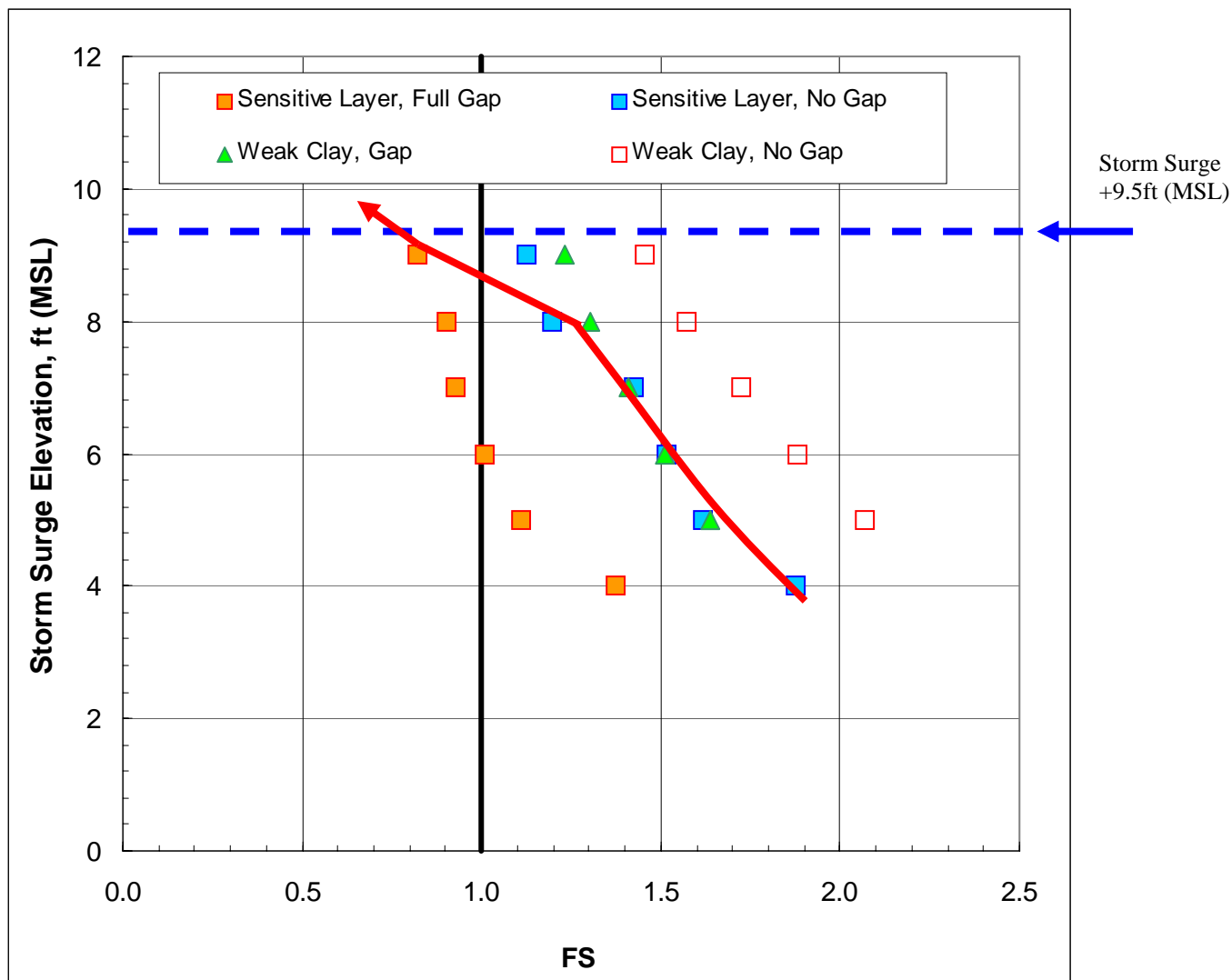


Figure 8.68: Factor of safety for four modes of failure at the 17th Street Canal breach site as a function of canal water elevation based on static Limit Equilibrium slope stability analyses: again showing the best-estimated path to failure.

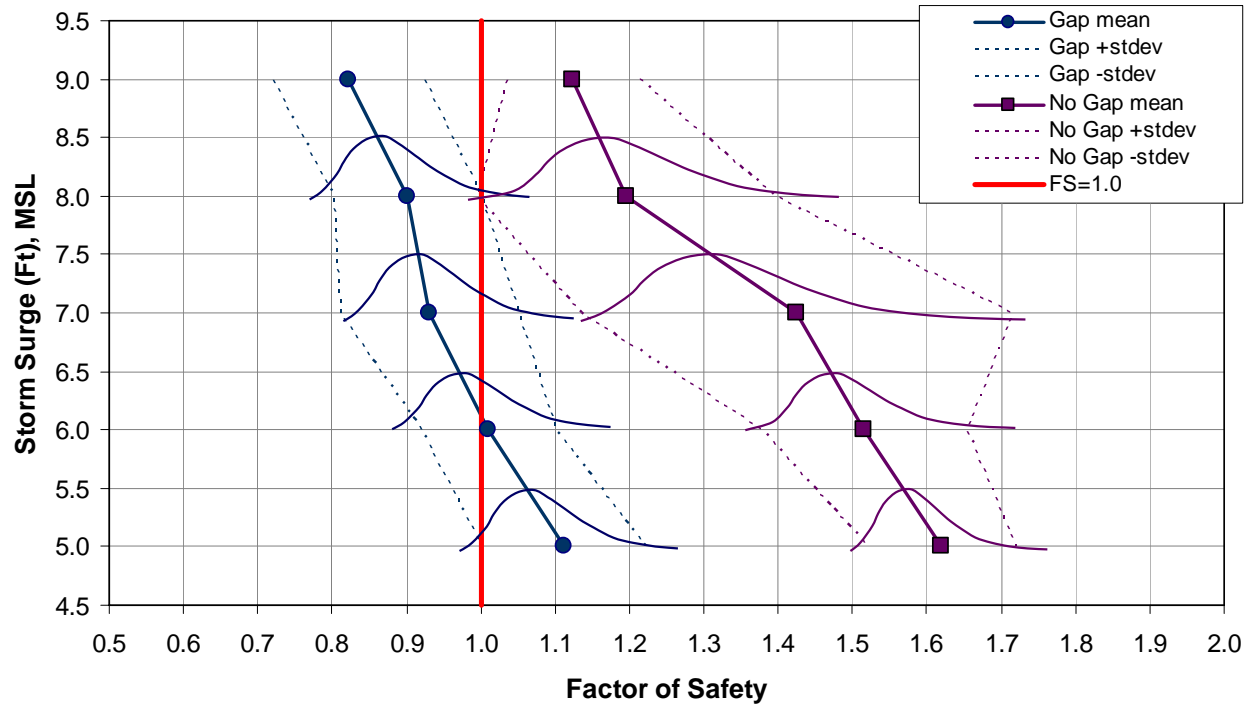


Figure 8.68(a): Distributions of Factor of Safety for the Ungapped and Water-Filled Gap Cases as a function of increasing canal water level; 17th Street Drainage Canal, east bank breach section.

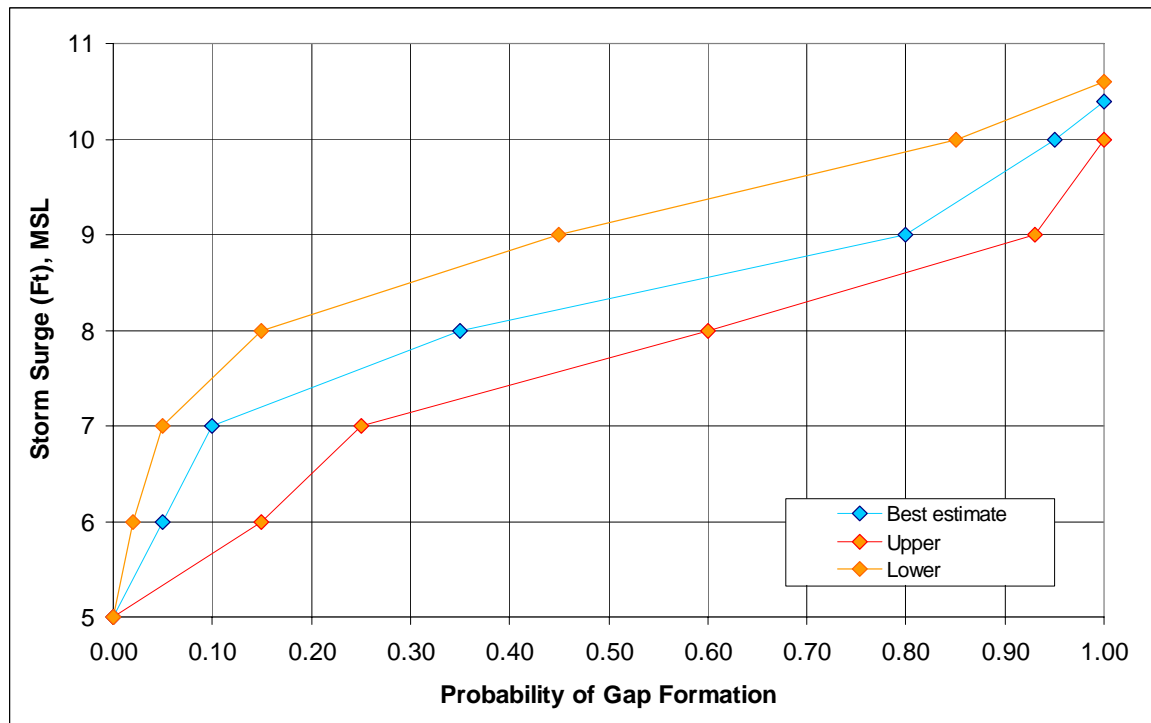


Figure 8.68(b): Estimated probabilities of formation of a significant water-filled gap as a function of increasing canal water level; 17th Street Drainage Canal, east bank breach section.

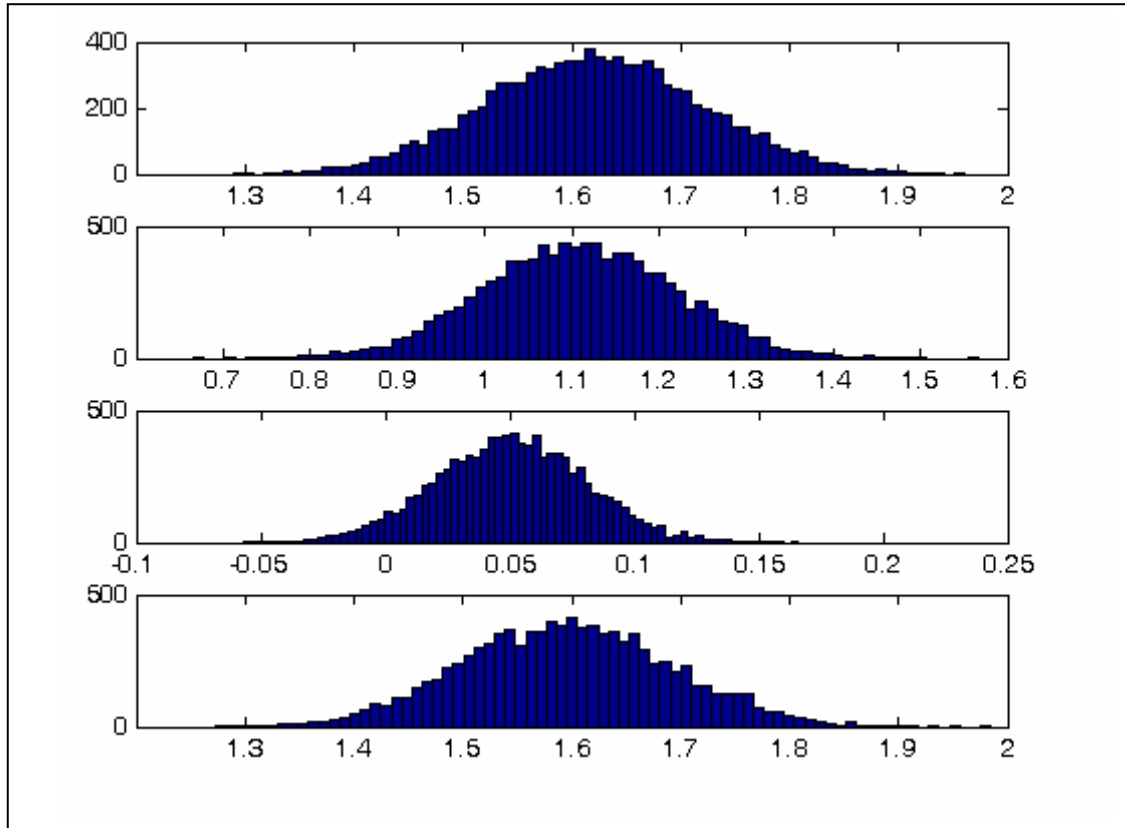


Figure 8.68(c): Example calculation (Monte Carlo simulation) for a canal water elevation of +5 feet (MSL).

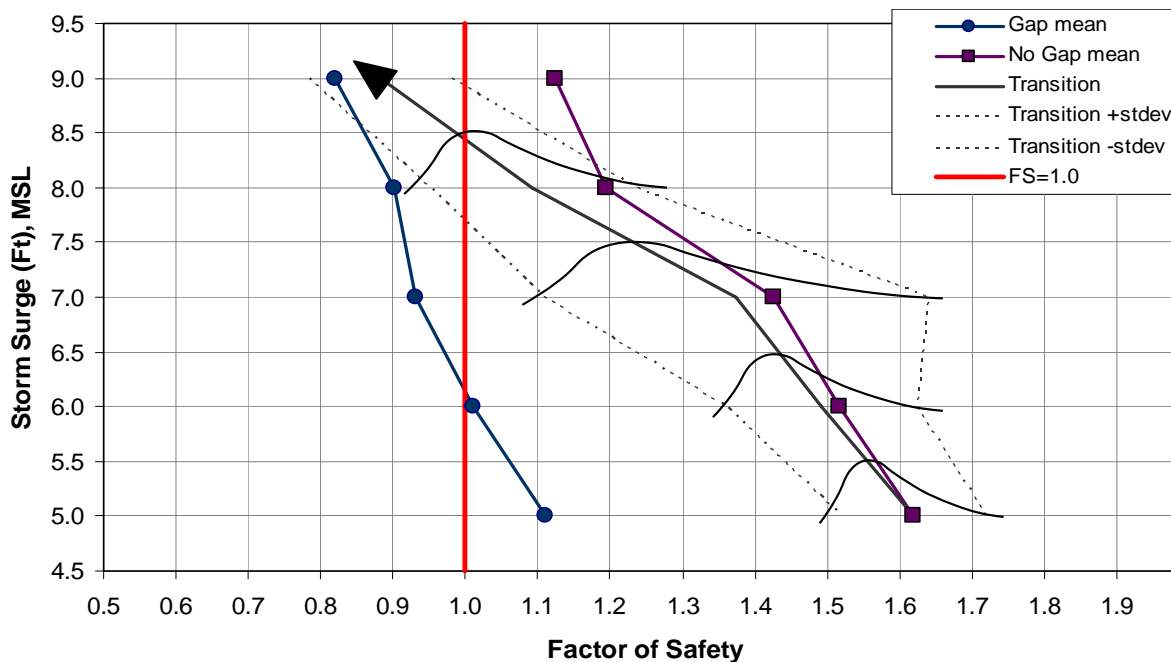


Figure 8.68(d): Distribution of Factor of Safety vs. canal water level with progressive transition from ungapped to water-filled gap conditions; 17th Street Canal, east bank breach section.

Storm Surge Elevation (ft), MSL	Probability of Failure
9	88.1%
8	25.6%
7	7.9%
6	0.01%
5	0.00%

Table 8.1: Probability of failure for 17th Street Canal, East Bank

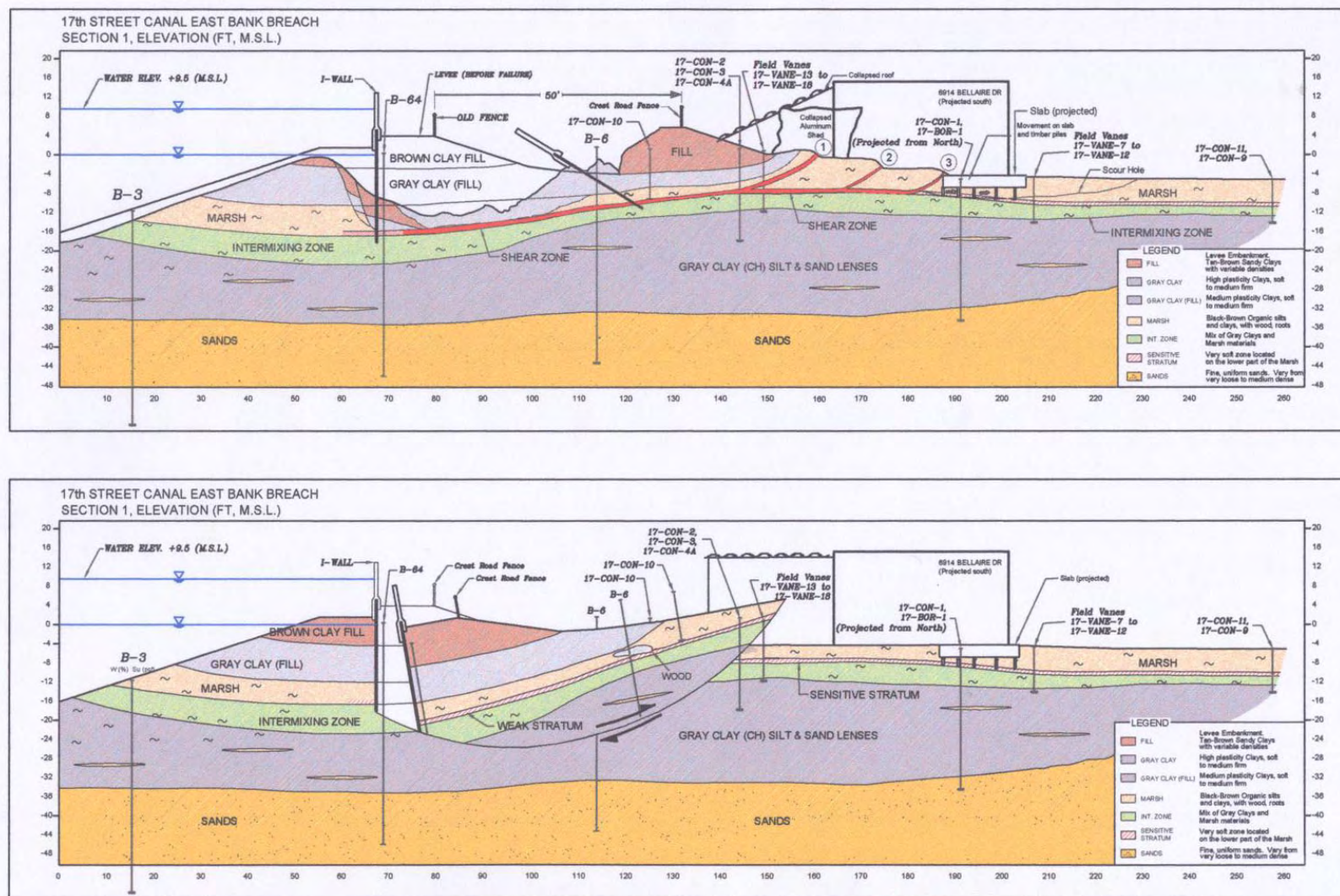
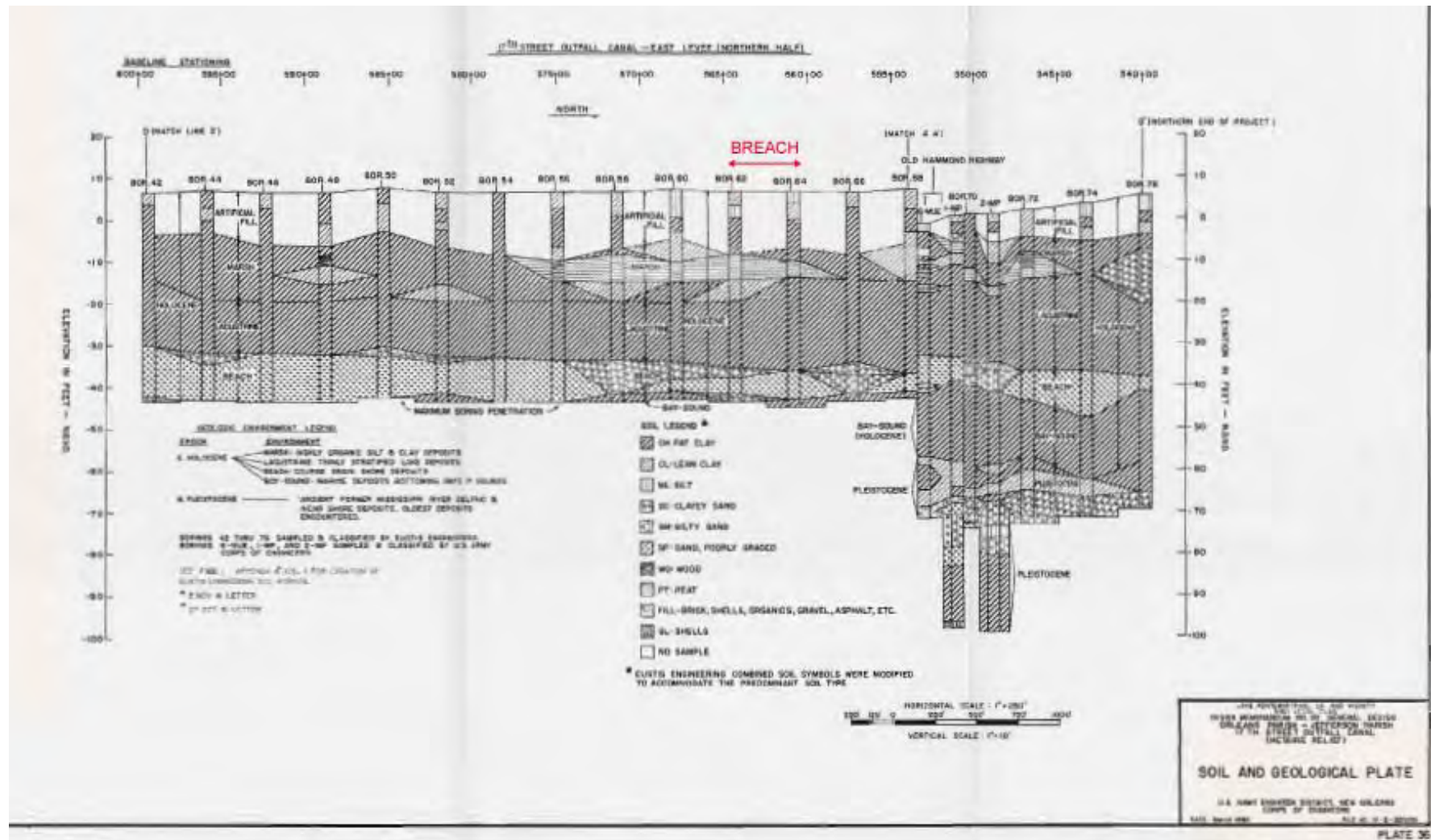


Figure 8.69: Comparison between deep rotational failure through the soft gray clay (CH) and translational failure along the sensitive clay layer within the marsh deposits as actually observed.



Initial longitudinal subsurface profile used for initial design at the 17th Street Canal breach site.

Figure 8.70:

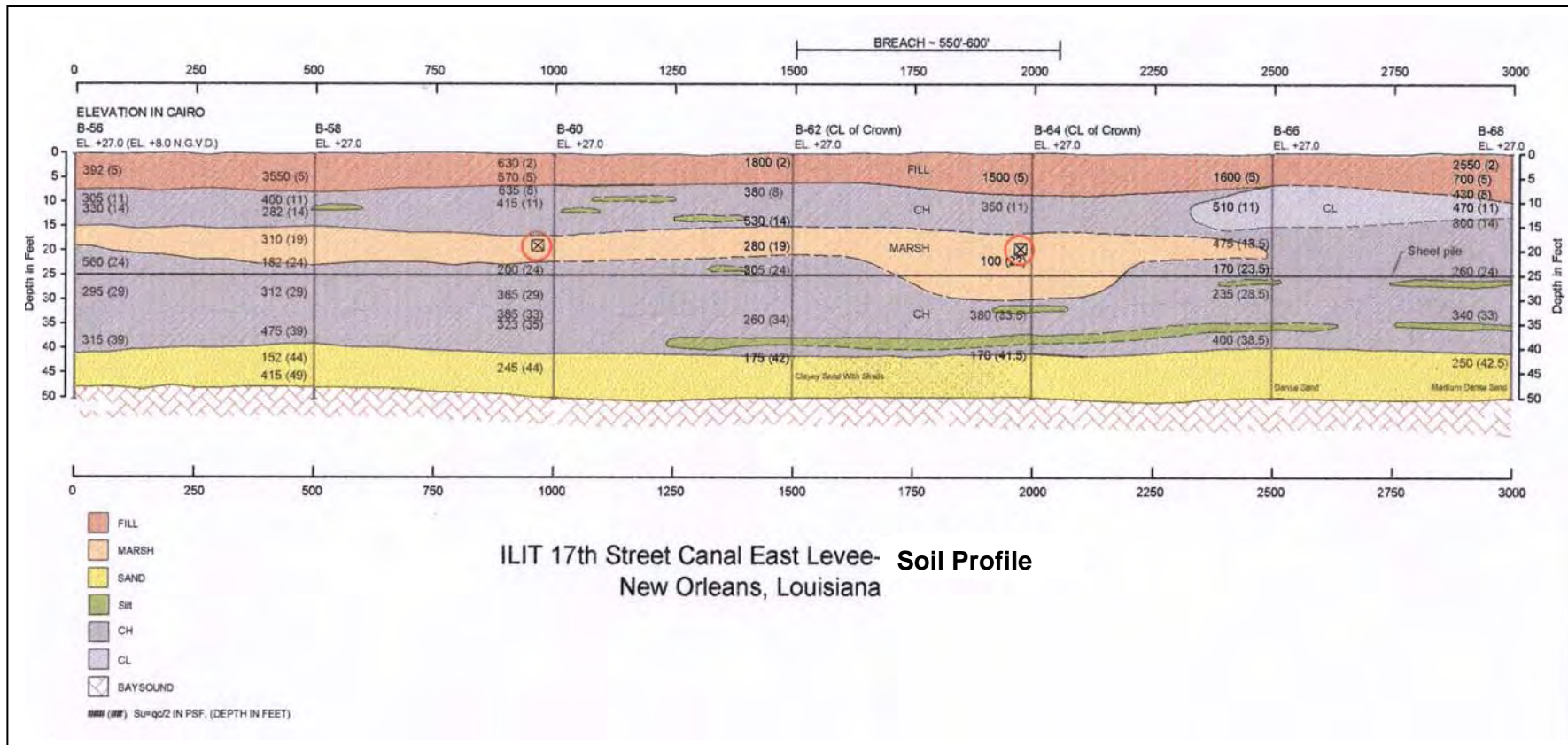


Figure 8.71: Re-interpreted longitudinal subsurface soil profile, showing location of breach section on the east side of the 17th Street Canal, and with non-tested samples highlighted.



[USACE, DM-20, Vol.1, 1990]

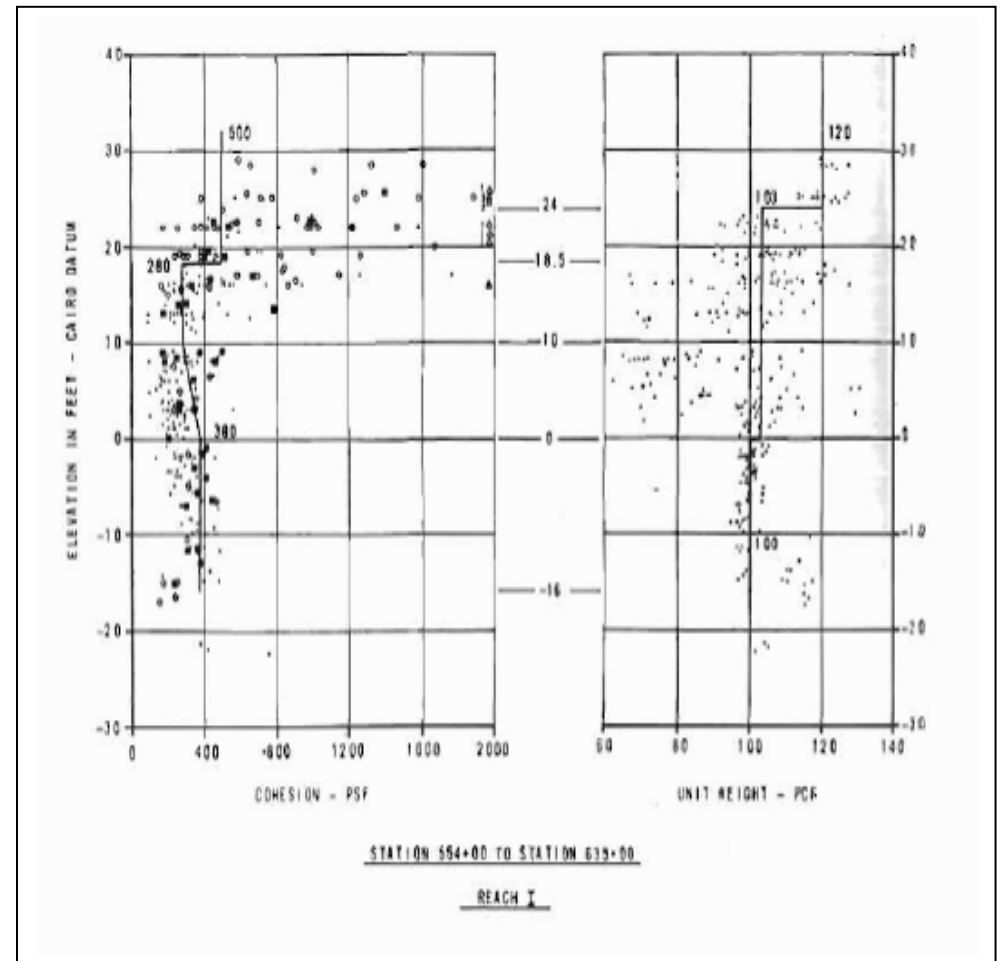
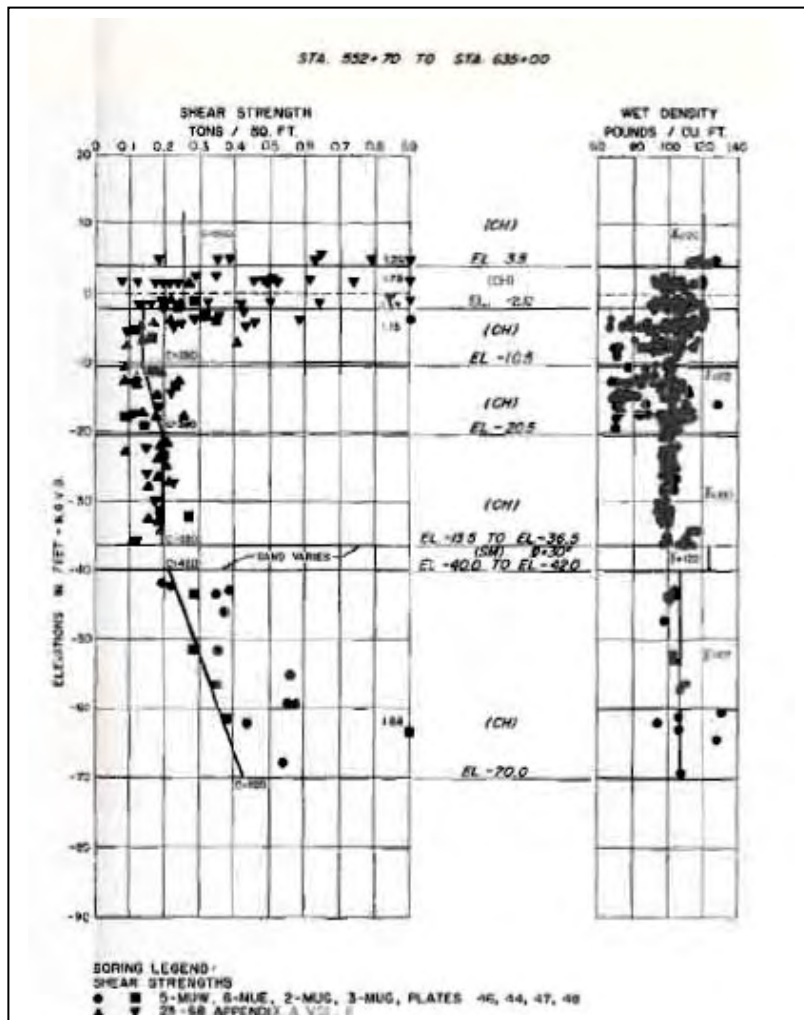


Figure 8.73: Profile of shear strength vs. depth used in original stability analyses for design at the 17th Street Canal breach site.
[USACE, DM-20, Vol.1 & 2, 1990]

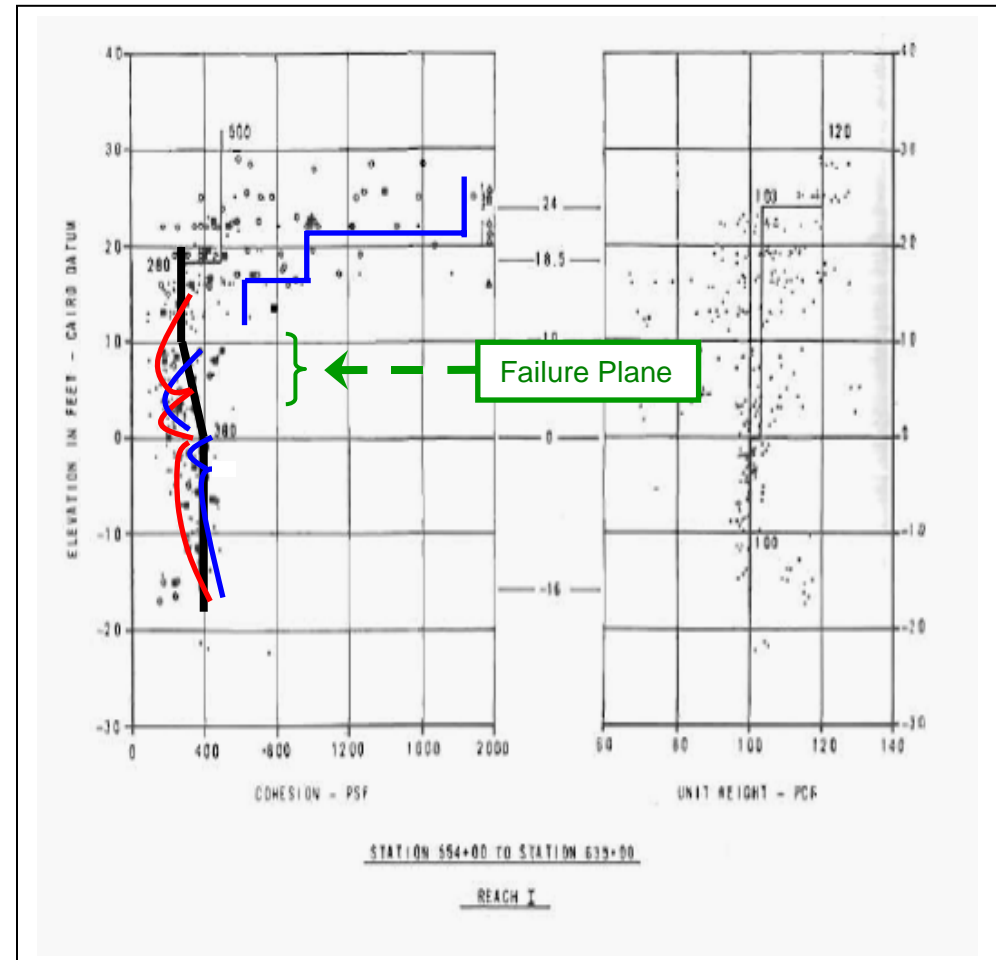
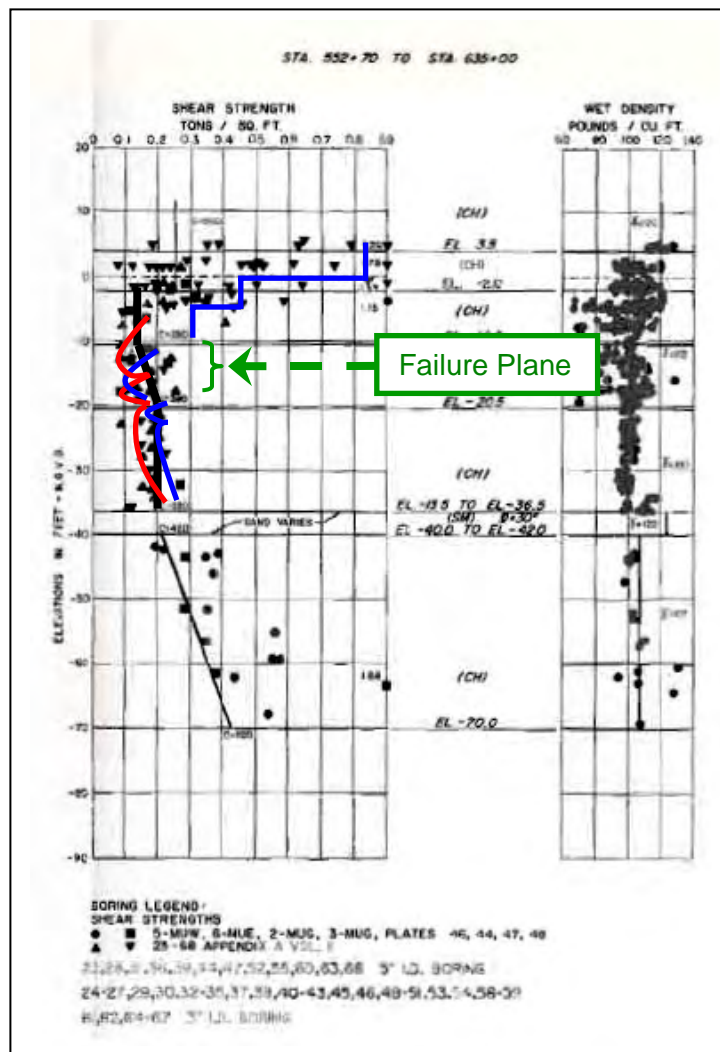


Figure 8.74: Profile of shear strength vs. depth used in original stability analyses for design at the 17th Street Canal breach site, and this investigation team's best estimated profiles of undrained shear strength vs. elevation (a) beneath levee crest [blue line], and (b) beneath levee toe [red line].

[USACE, DM-20, Vol.1 & 2, 1990]



Figure 8.75: View of floodwall displacement on the west bank of the 17th Street Canal.

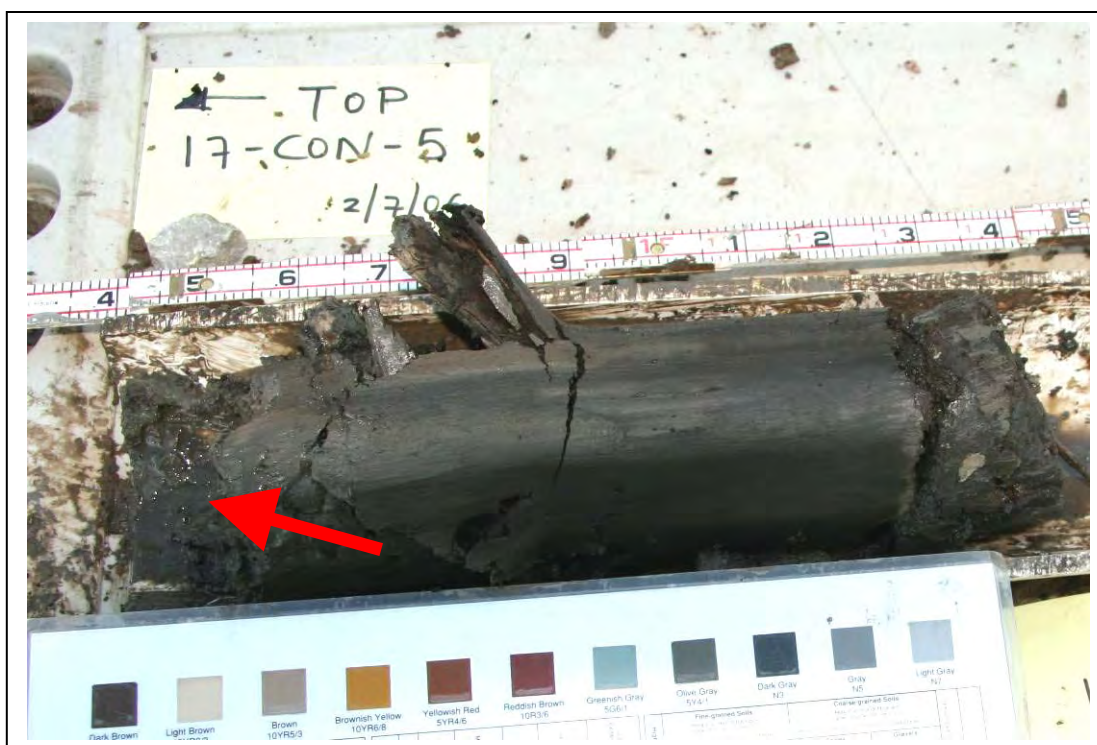


Figure 8.76: View of sample of “sensitive” clay layer within the marsh soils at the west side of the 17th Street Canal.

17th STREET CANAL WEST BANK BREACH ANALYSIS SECTION

Elevation (ft. N.A.V.D.)

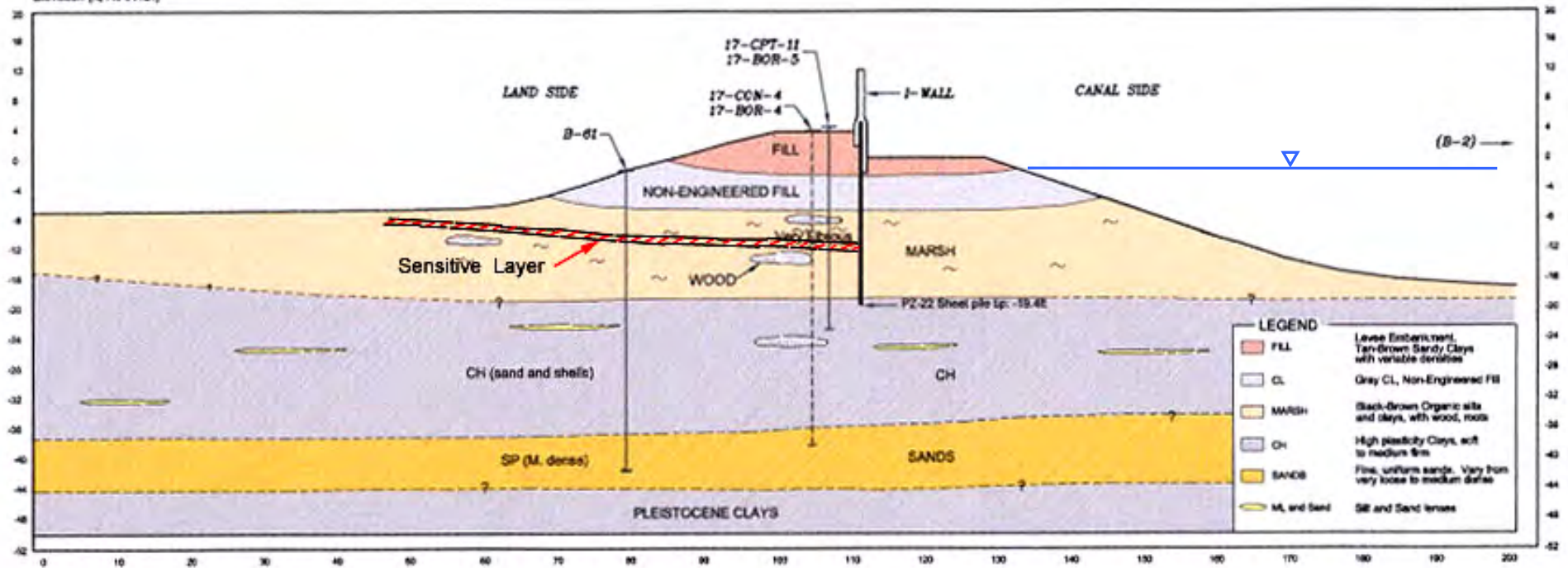


Figure 8.77: Analysis cross-section for the site on the west bank of the 17th Street Canal.

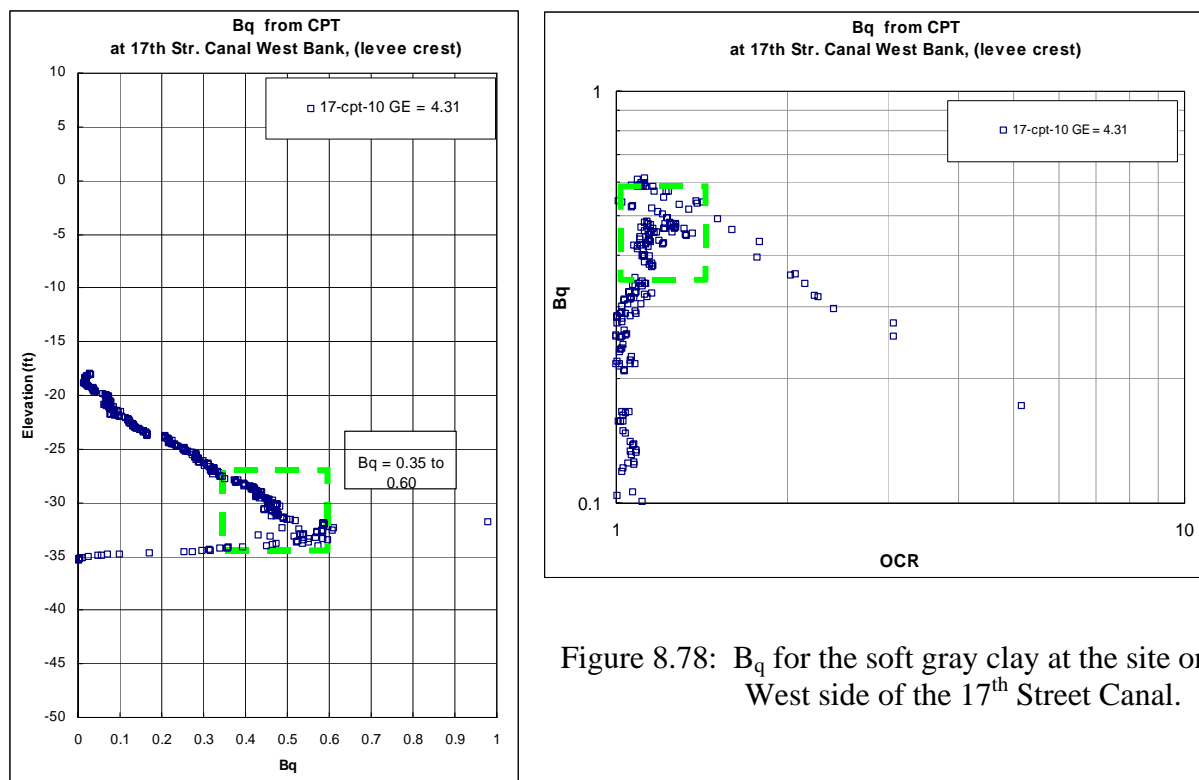


Figure 8.78: B_q for the soft gray clay at the site on the West side of the 17th Street Canal.

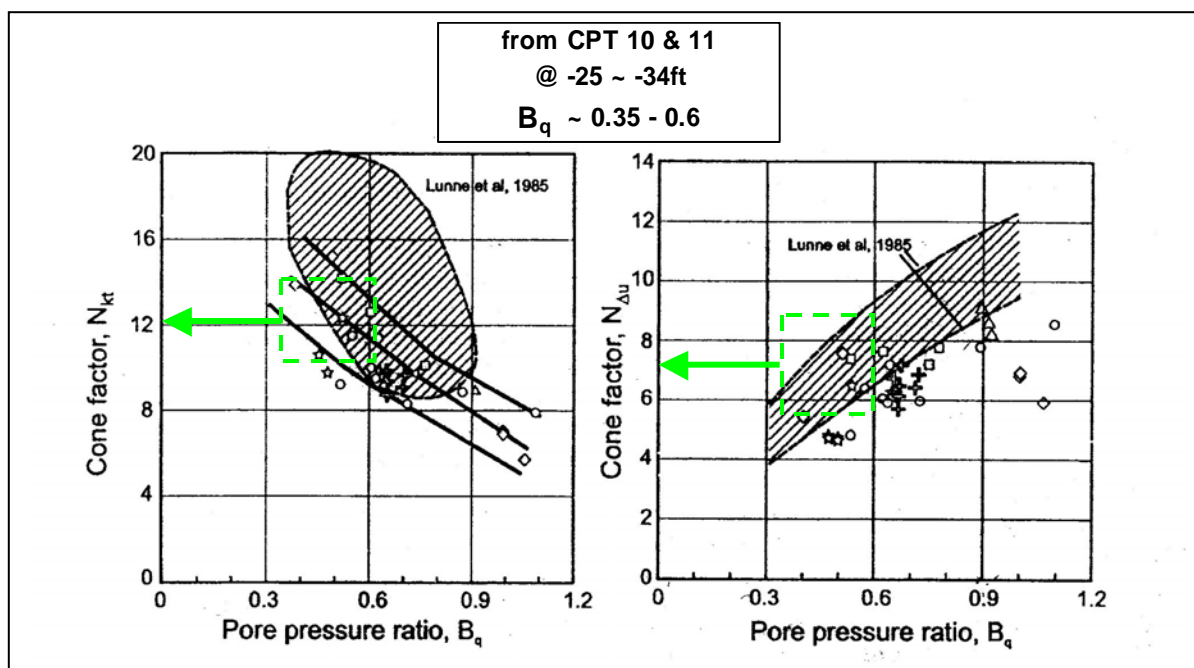
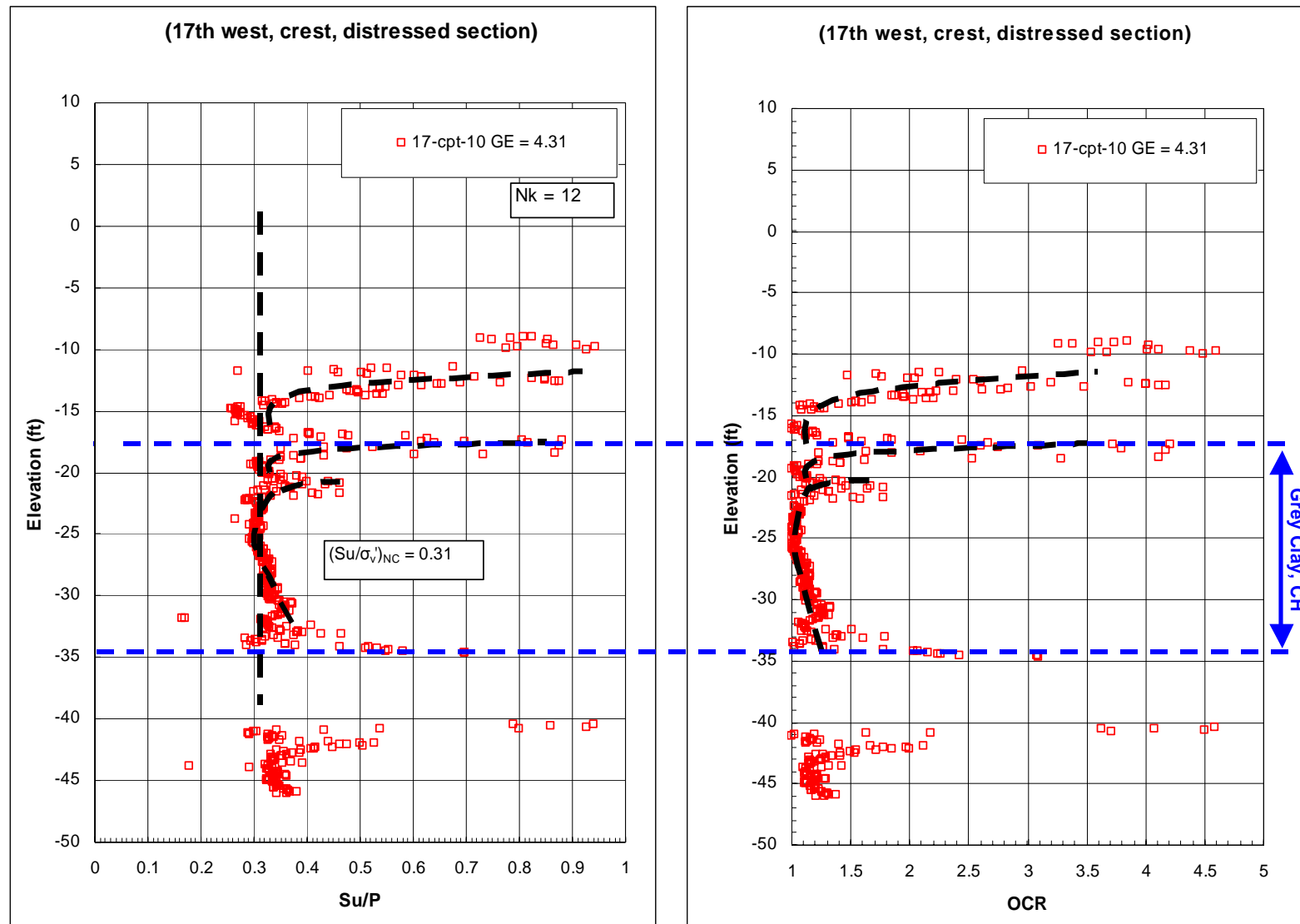


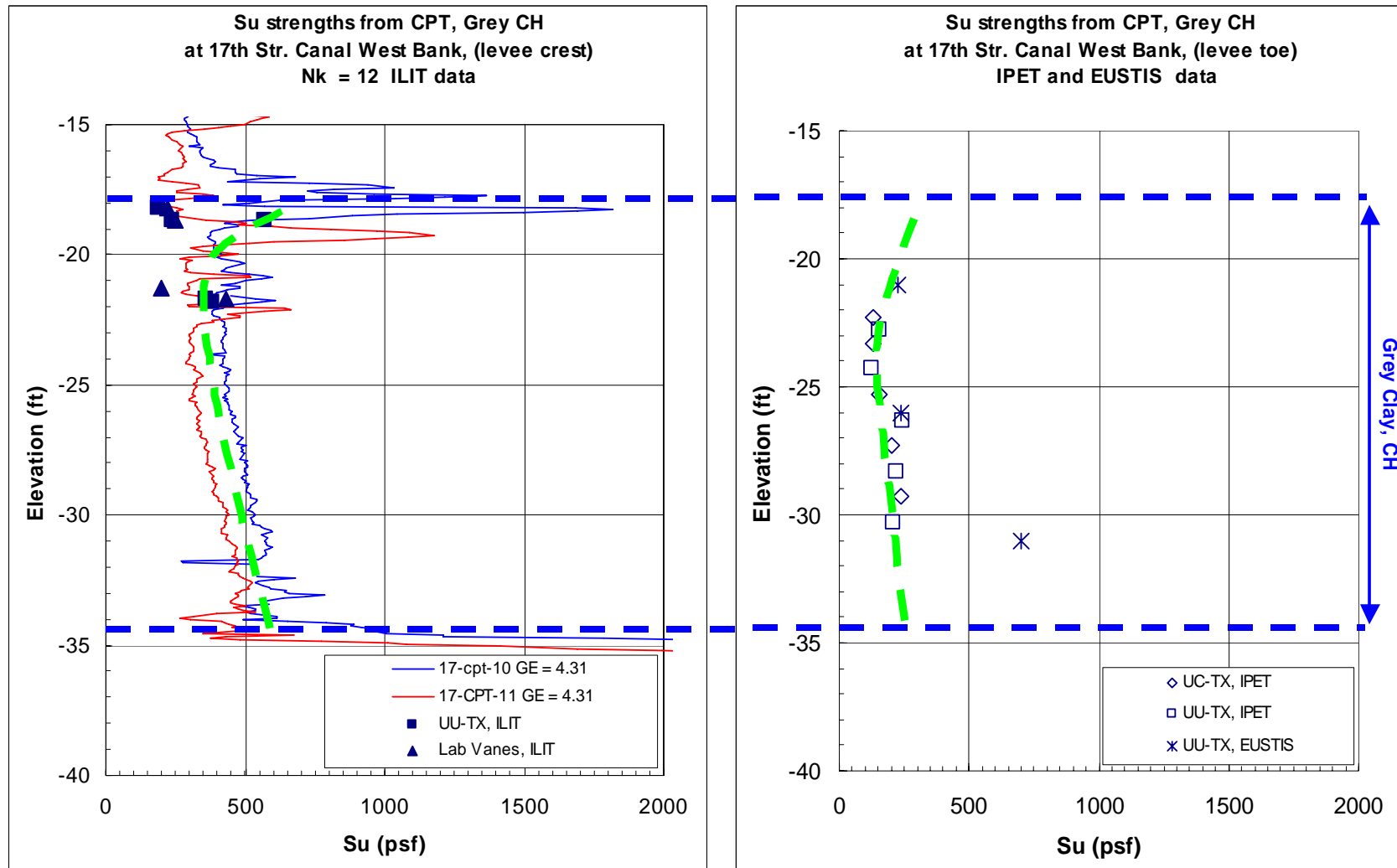
Figure 8.79: N_{kt} and N_{du} for the soft gray clay at the site on the west side of the 17th Street Canal based on CPTU.



S_u/P vs. depth and OCR vs. depth for soft gray clay (CH) beneath the crest of the embankment for the site on
th Street Canal.

Figure 8.80:

the west side of the 17



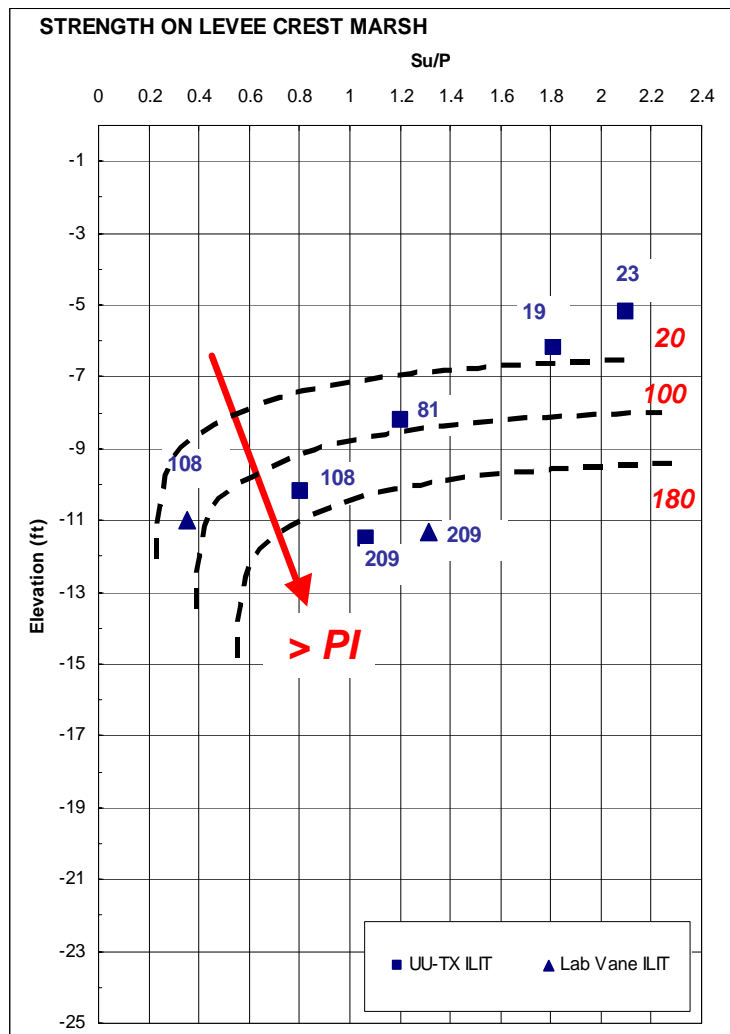
(a) Beneath the levee crest

(b) Beneath the inboard of the levee toe

(a) Shear strength vs. depth within the soft gray clay (CH) for the site on the west side of the 17th Street Canal (a) beneath the crest of the levee, and (b) beneath the inboard side toe of the levee embankment.

Figure 8.81:

Ref: Mayne, P.W and Mitchell,
J.K (1988)



elev (ft)	$S_u/\sigma'v$			OCR= $\alpha_{VST} (S_u/\sigma'v)$		
	PI (%)			$\alpha_{VST} = 22 PI^{0.48}$		
	20	100	180	5.2	2.4	1.8
-5	2.4			12.5		
-6	2.2			11.5		
-7	1.2			6.3		
-8	0.6	2.2		3.1	5.3	
-9	0.34	0.9	3	1.8	2.2	5.5
-10	0.27	0.6	1.4	1.4	1.4	2.5
-11	0.22	0.4	0.8	1.1	1.0	1.5
-12	0.2	0.4	0.6	1.0	1.0	1.1
-13	0.2	0.4	0.55	1.0	1.0	1.0
-14	0.2	0.4	0.55	1.0	1.0	1.0

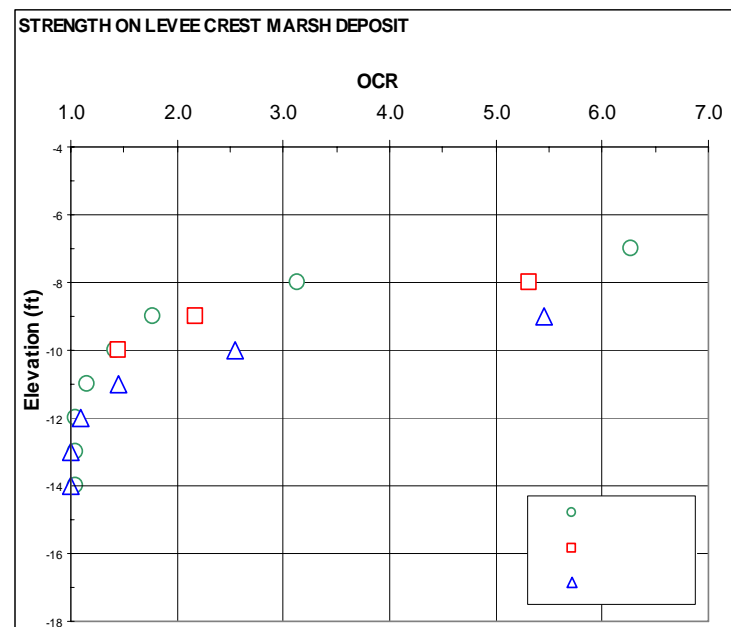


Figure 8.82: S_u/P and OCR estimation from PI and vane shear tests for the marsh deposits at the th Street Canal.

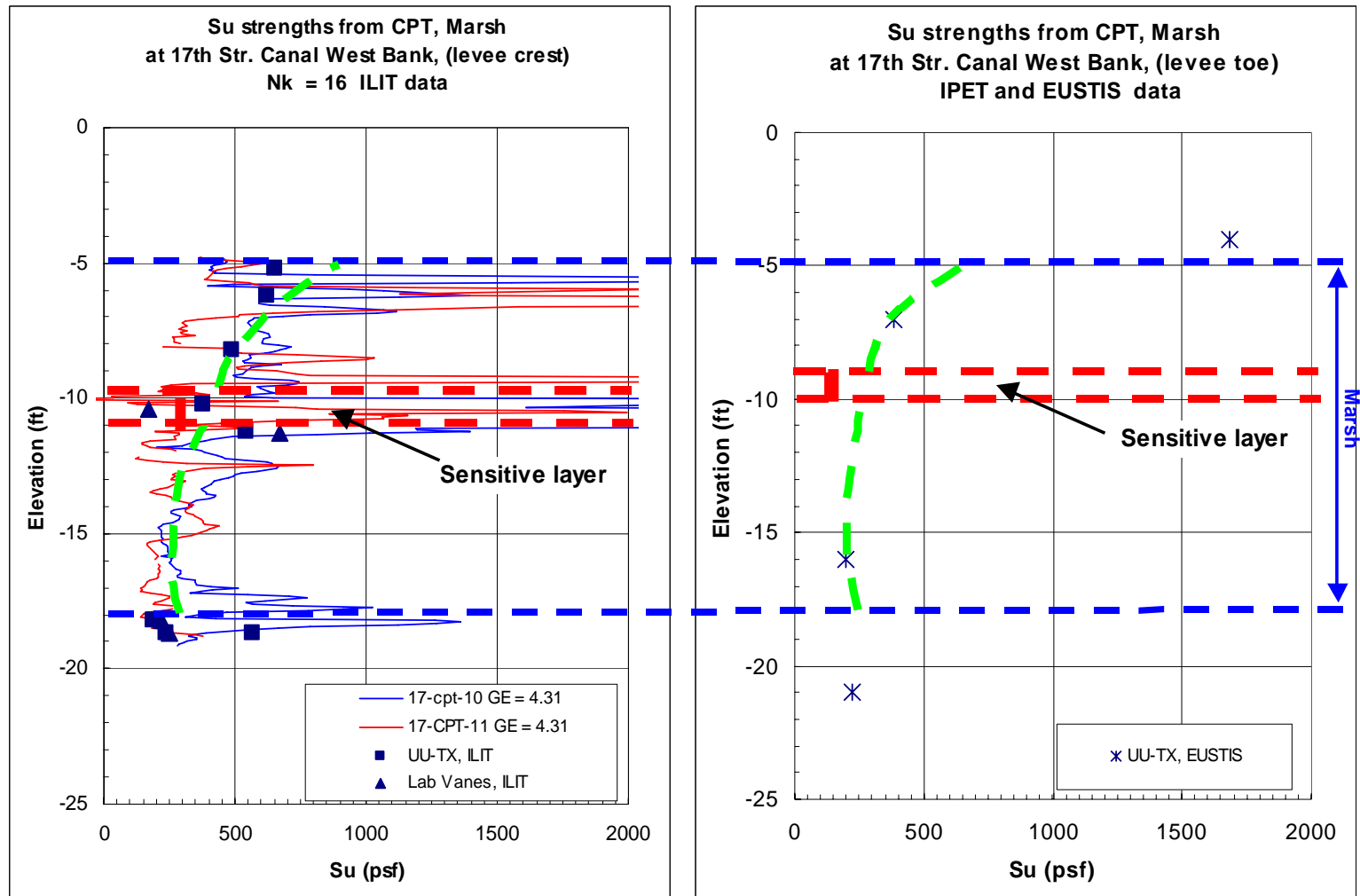


Figure 8.83: Shear strength vs. depth within the marsh deposits at the site on the west side of the 17th Street Canal (a) beneath the crest of the levee, and (b) beneath the inboard toe of the embankment.

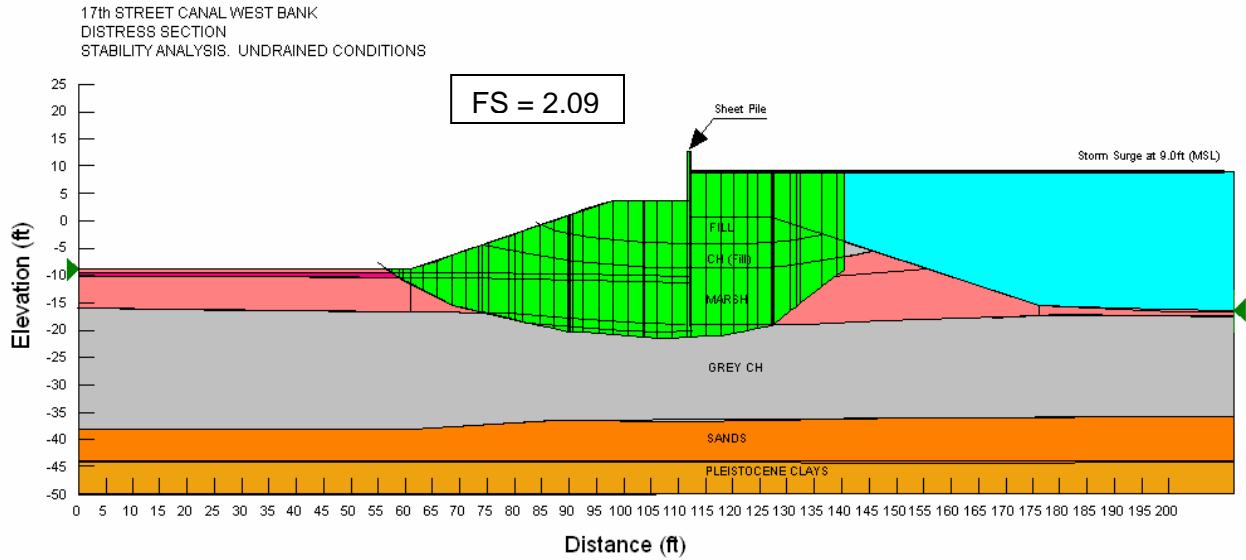


Figure 8.84: Stability analysis of the west side of the 17th Street Canal with water at Elev. + 9 feet (MSL) for case of rotational failure through the soft gray clay (CH) with no gap developed.

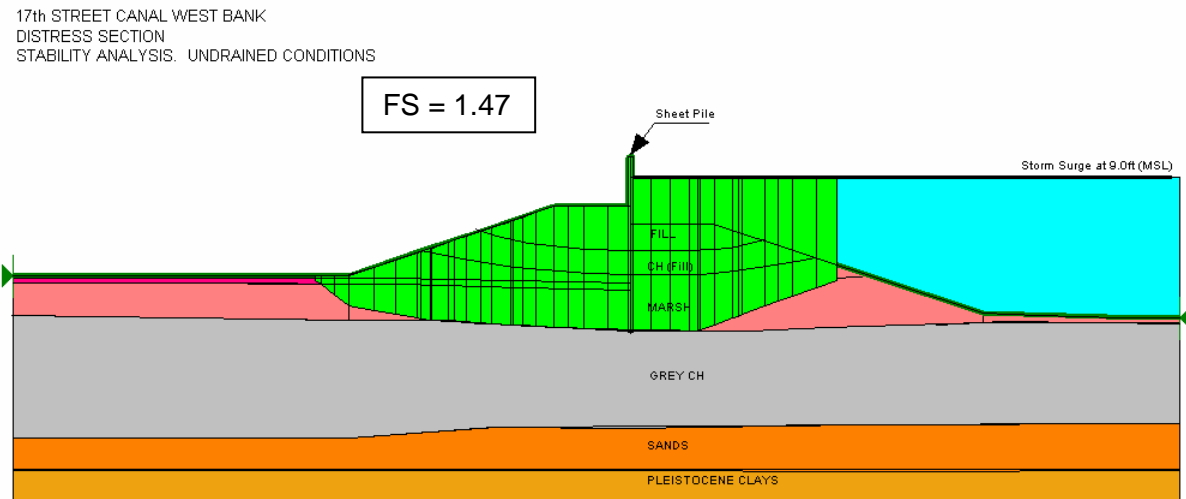


Figure 8.85: Stability analysis of the west side of the 17th Street Canal with water at Elev. + 9 feet (MSL) for case of failure through the base of the “marsh” layer (with no gap developed).

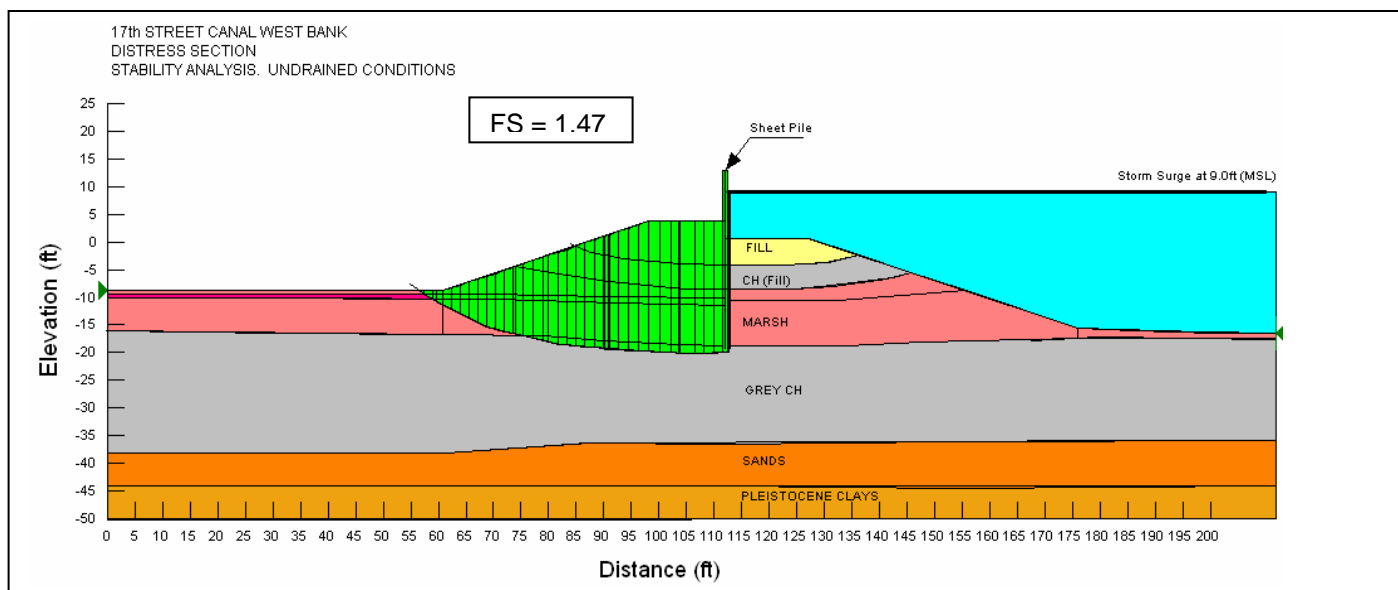


Figure 8.86: Stability analysis of the west side of the 17th Street Canal with water at Elev. + 9 feet (MSL) for case of rotational failure through the soft gray clay (CH) (with water-filled gap).

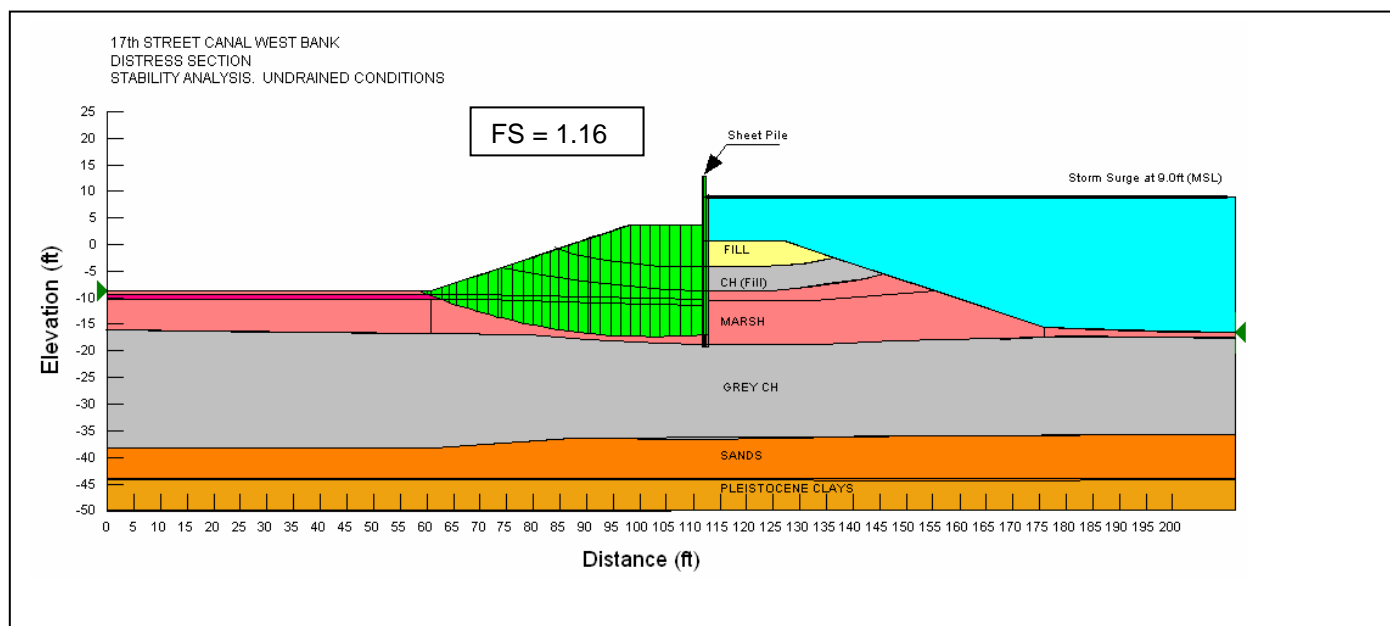


Figure 8.87: Stability analysis of the west side of the 17th Street Canal with water at Elev. + 9 feet (MSL) for case of failure along the base of the “marsh” layer (with water-filled gap).

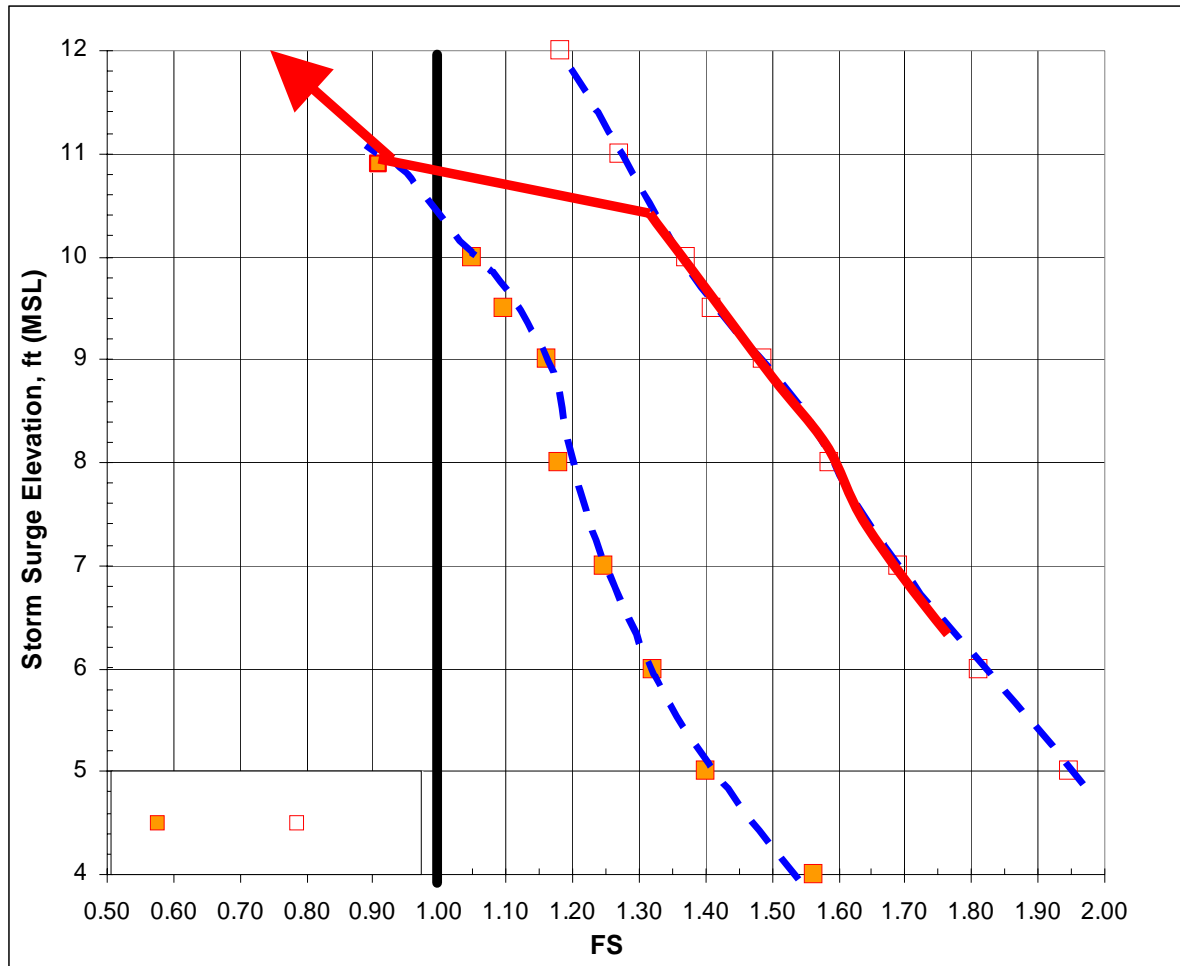


Figure 8.88: Factor of safety vs. water level on the west bank of the 17th Street Canal.

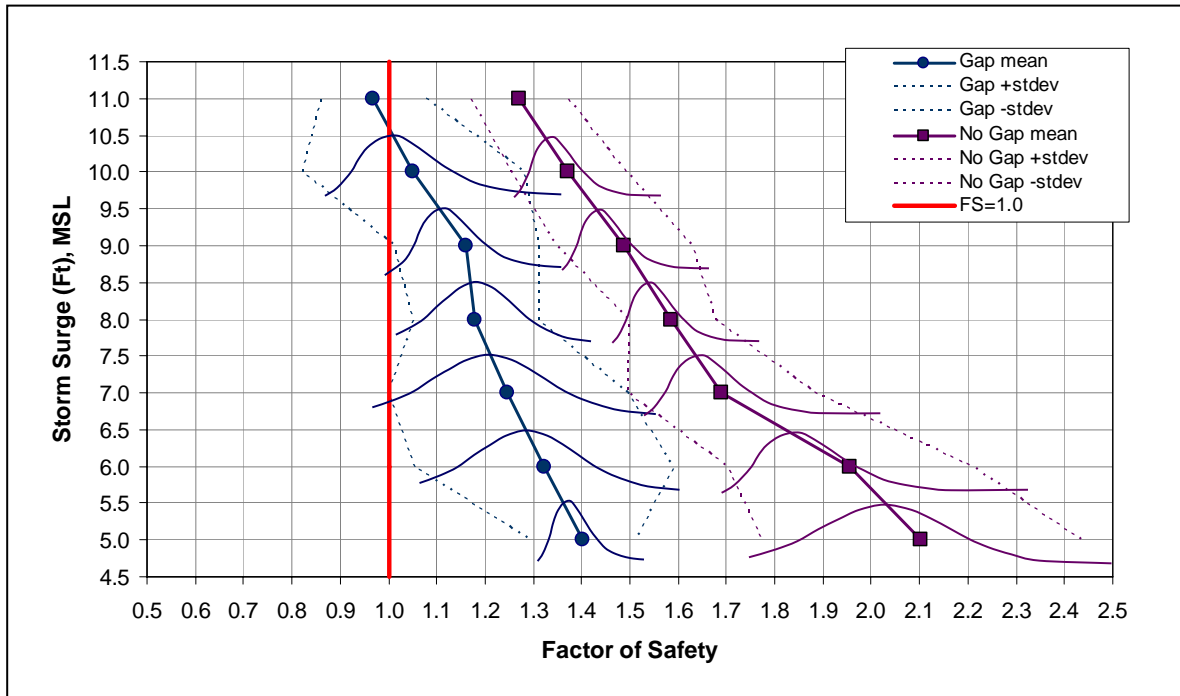


Figure 8.88(a): Distributions of factors of safety for the “Water-filled Gap” and the “Ungapped” cases as a function of rising canal water elevations; 17th Street Drainage Canal; west bank distressed section.

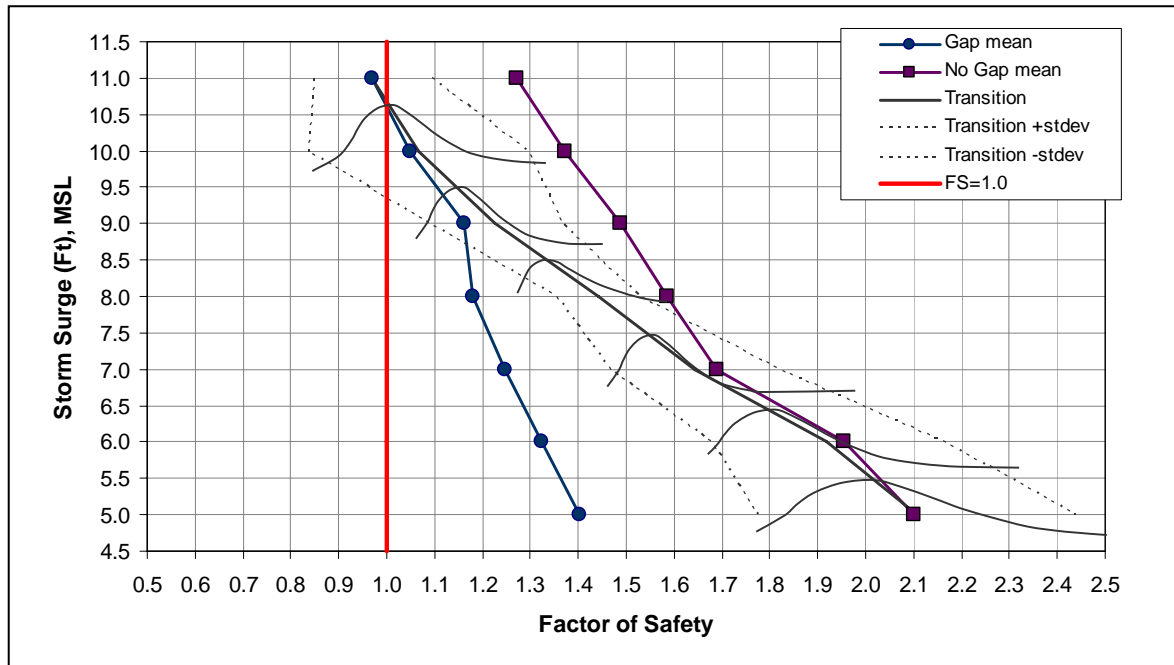


Figure 8.88(b): Distributions of conjugate overall factors of safety for the 17th Street Drainage Canal; west bank distressed section.

Storm Surge Elevation (ft), MSL	Probability of Failure
11	59.2%
10	38.7%
9	4.9%
8	0.10%
7	0.013%
6	0.007%
5	0.0%

Table 8.2: Probability of failure for 17th Street Canal, West Bank



Figure 8.89: View of failure and breach on the east bank near the south end of the London Avenue Canal.



Figure 8.90: View of sands piled in neighborhood inboard of the south London Avenue Canal breach.



Figures 8.91 and 8.92: Views of floodwall panels “dropping” into the eroded void at the north and south ends of the south London Avenue canal breach.



Figure 8.93: View of trees at the inboard levee toe immediately to the north of the end of the south breach in the London Avenue Canal.



Figure 8.94: View of toppled trees at the London Avenue Canal south breach site.

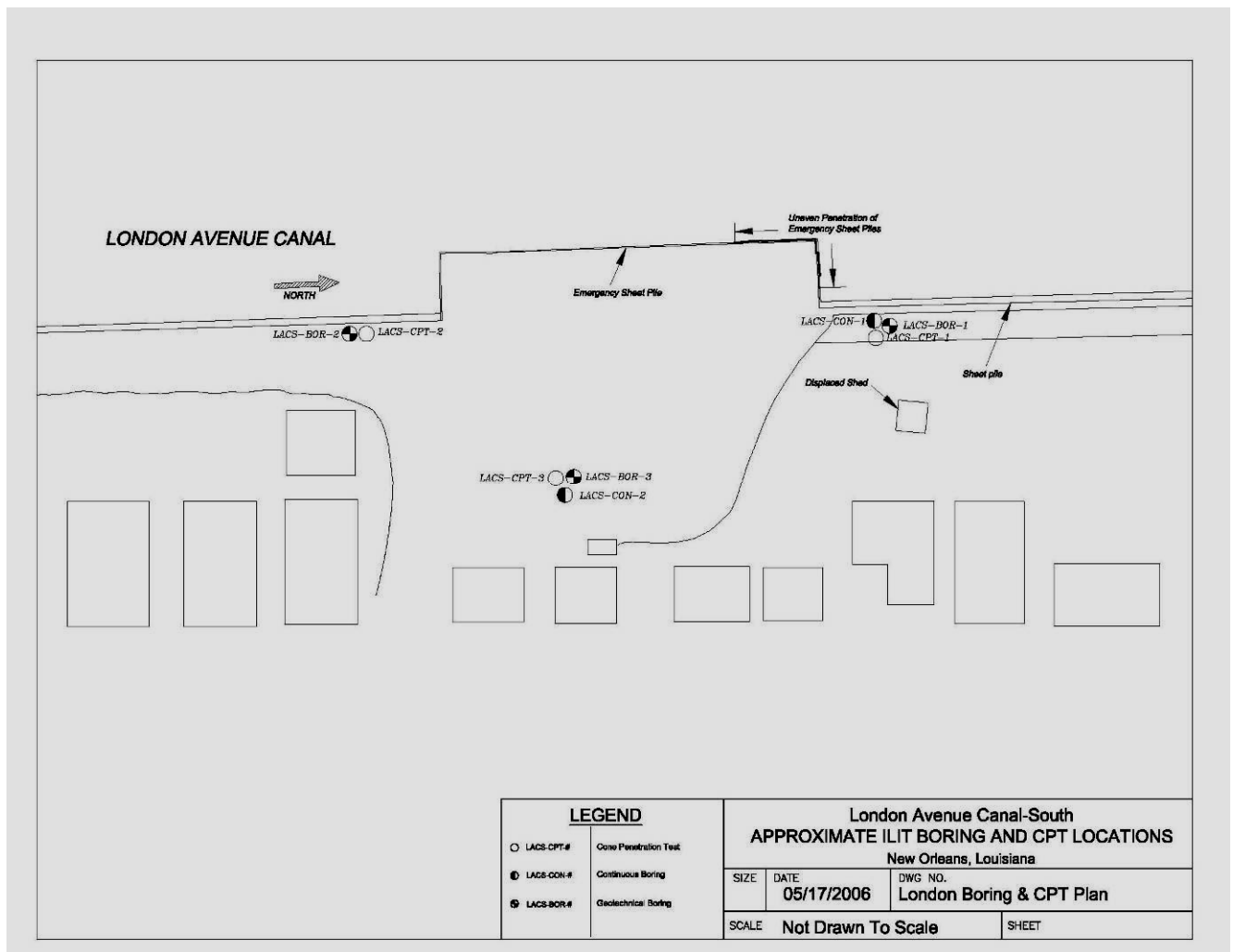


Figure 8.95: Plan view of the London Avenue Canal (South) breach site.

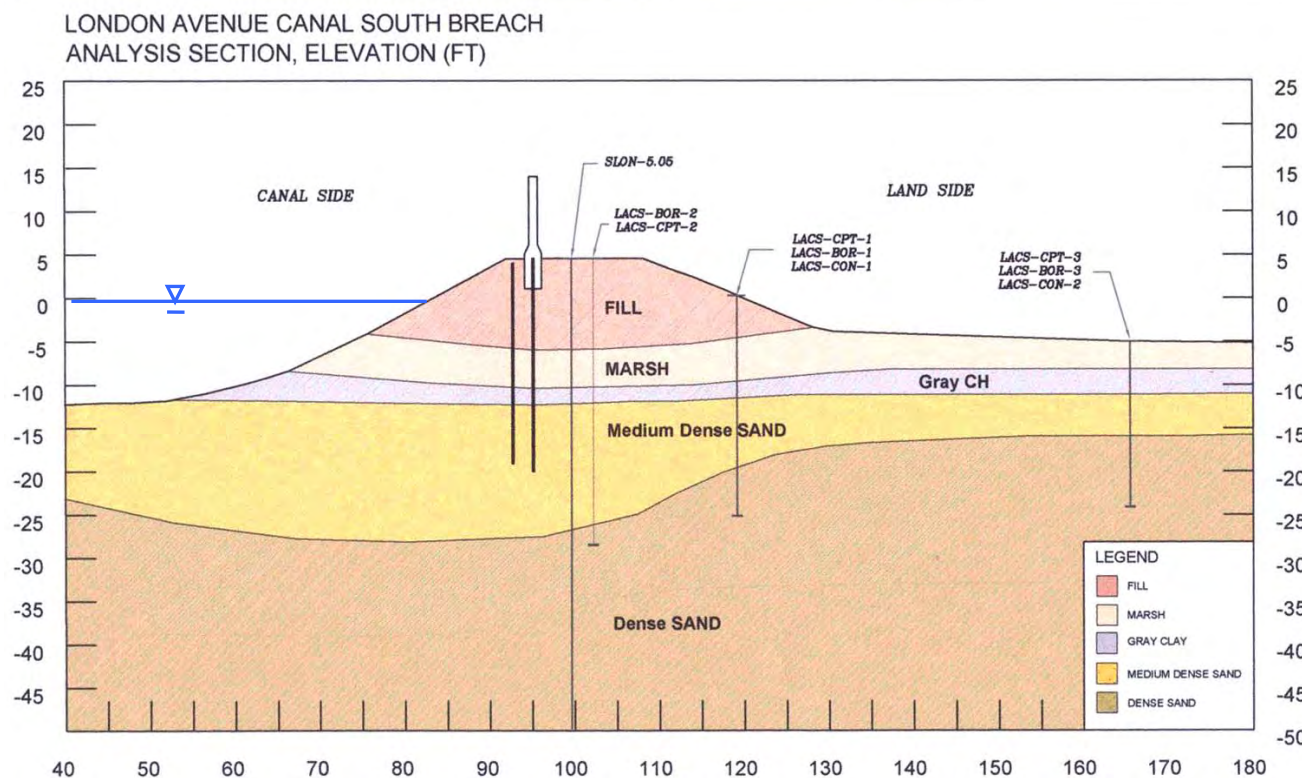
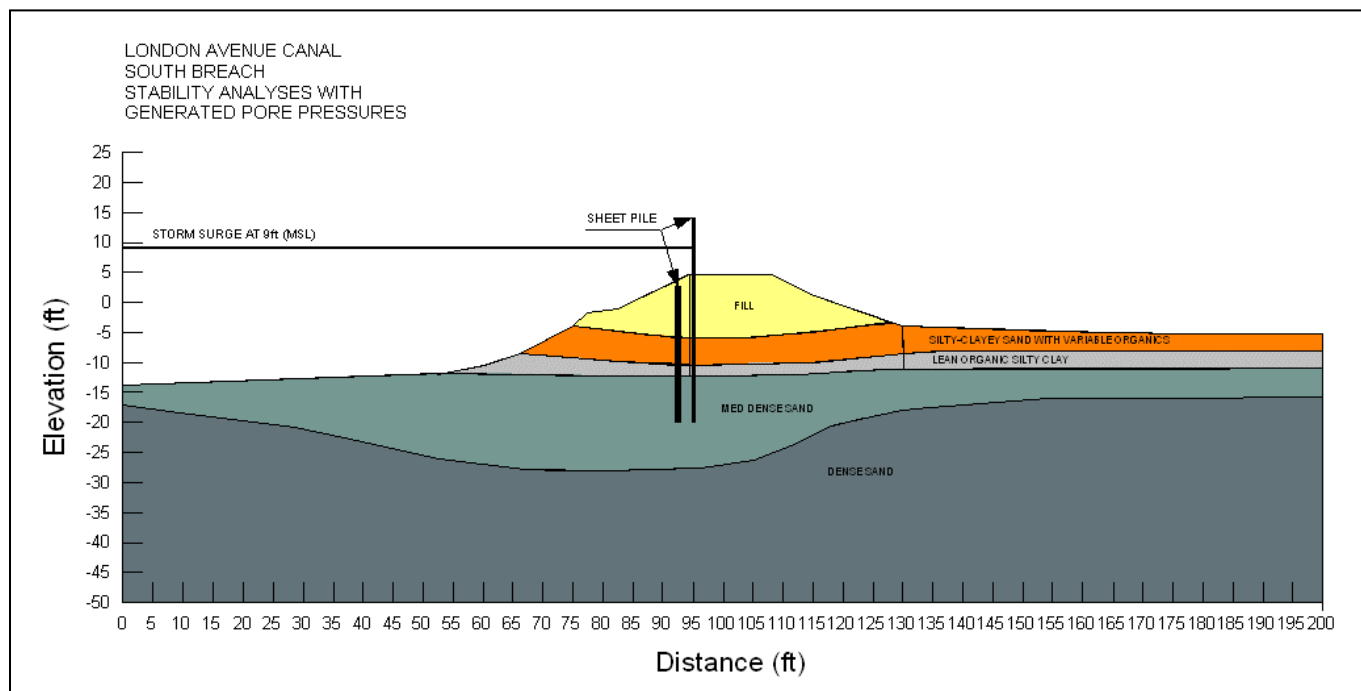


Figure 8.96: Cross-section through the breach near the south end of the London Avenue Canal.



ID	Soil Model	Type	γ	c	phi	k
			[lb/ft ³]	[lb/ft ²]	[°]	[ft/hr]
Upper fill	Mohr-Coulomb	Undrained	100	800	0	1.20E-03
SM/SC w/organics, crest	Mohr-Coulomb	Undrained	90	250	0	0.011
SM/SC w/organics, free field	Mohr-Coulomb	Undrained	90	200	0	0.011
Lean organic silty clay, crest	Mohr-Coulomb	Undrained	100	500	0	3.60E-03
Lean organic silty clay, free field	Mohr-Coulomb	Undrained	100	300	0	3.60E-03
Medium Dense Sand	Mohr-Coulomb	Drained	100	0	33	1.100
Dense Sand	Mohr-Coulomb	Drained	110	0	35	0.600

Figure 8.97: Geotechnical cross-section for analysis of the London Avenue south breach.

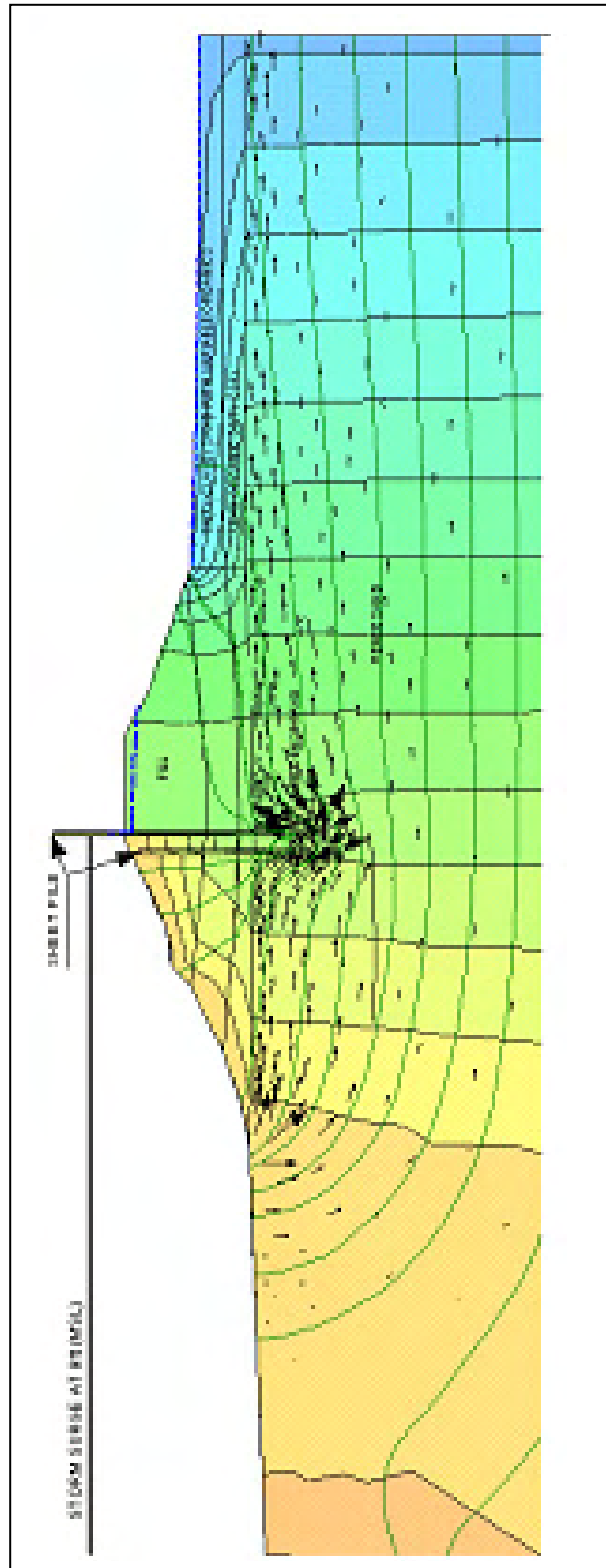


Figure 8.98: Flow vectors and head contours for seepage analysis of the south London Avenue Canal breach for a surge at Elev. + 9 feet (MSL).

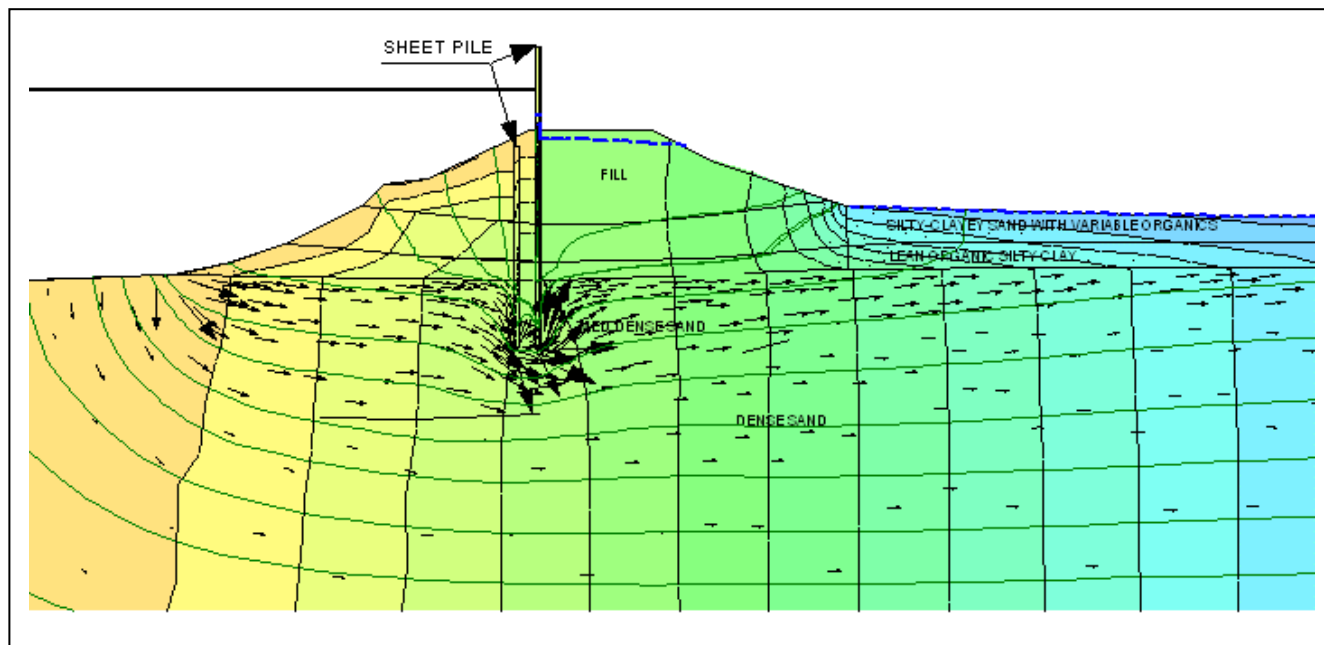


Figure 8.99: Closeup of flownet for storm surge at + 9ft (MSL) in the London Avenue Canal south breach. (Equipotential contours at intervals of one foot of head)

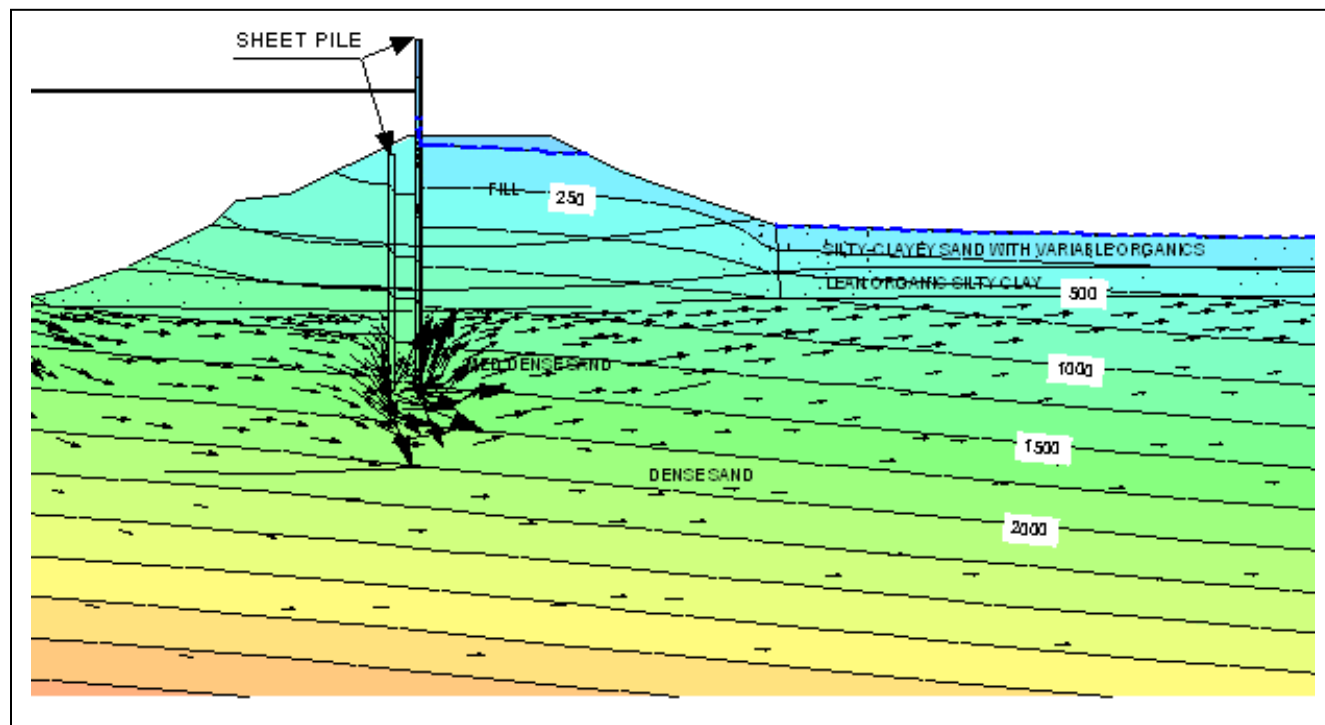


Figure 8.100: Pressure contours for Storm Surge at 9ft (MSL) in the London Avenue Canal South breach. (Pressure contours every 250 lb/ft²)

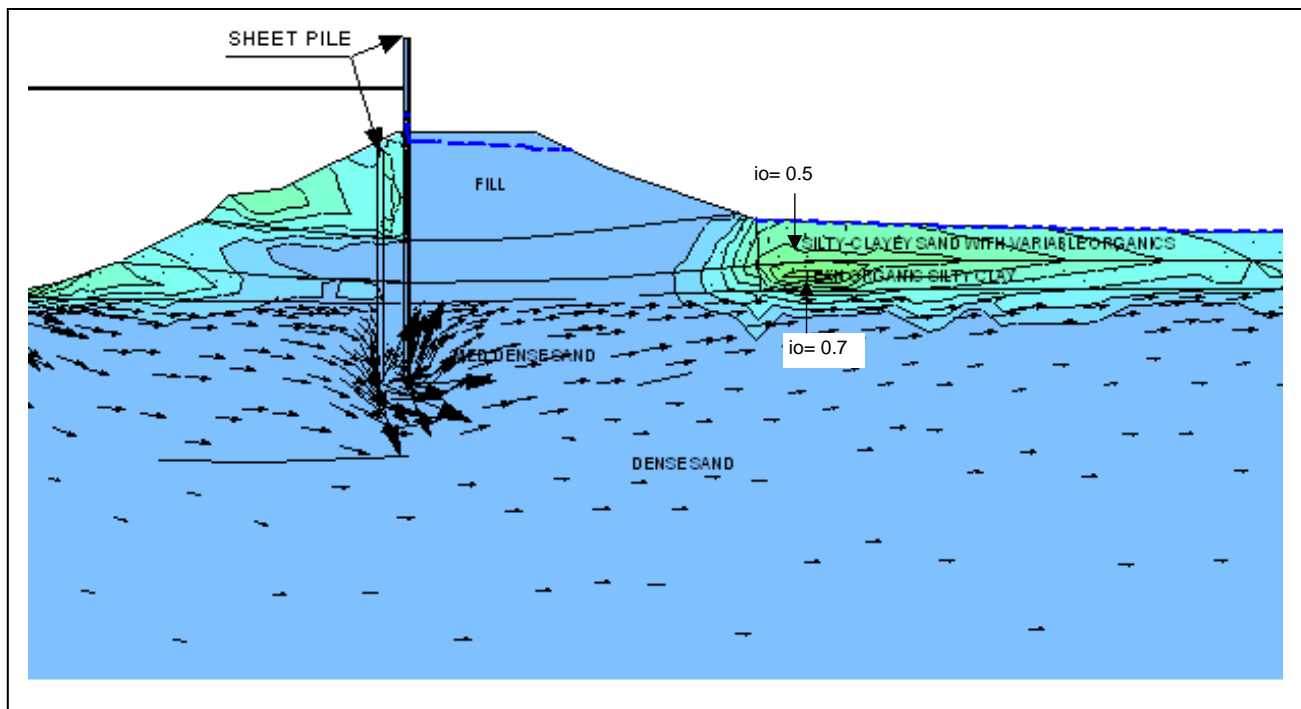


Figure 8.101: Hydraulic gradients for storm surge at +9ft (MSL) in the London Avenue Canal south breach. (Maximum toe exit gradient $i_o = 0.7$)

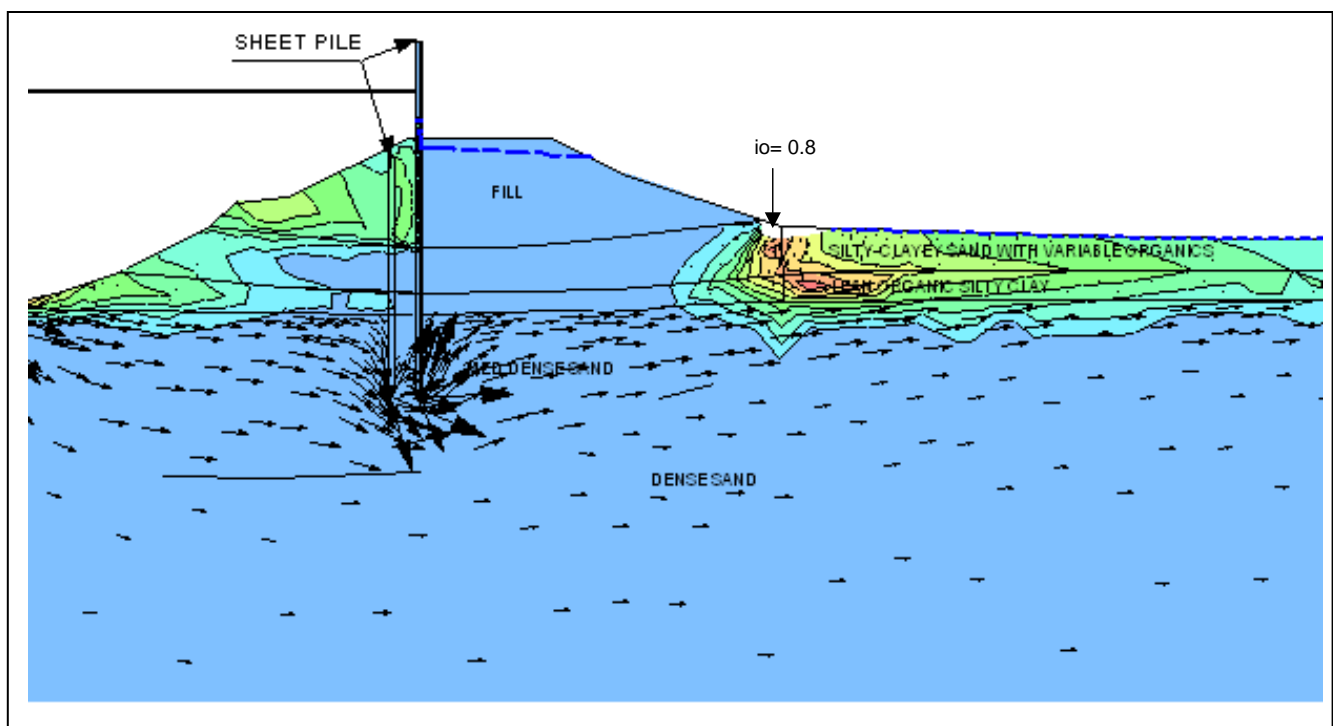


Figure 8.102: Hydraulic gradients for storm surge at +9ft (MSL) in the London Avenue Canal south breach: initiation of erosion at levee toe. (Maximum gradient $i_o = 0.8$)

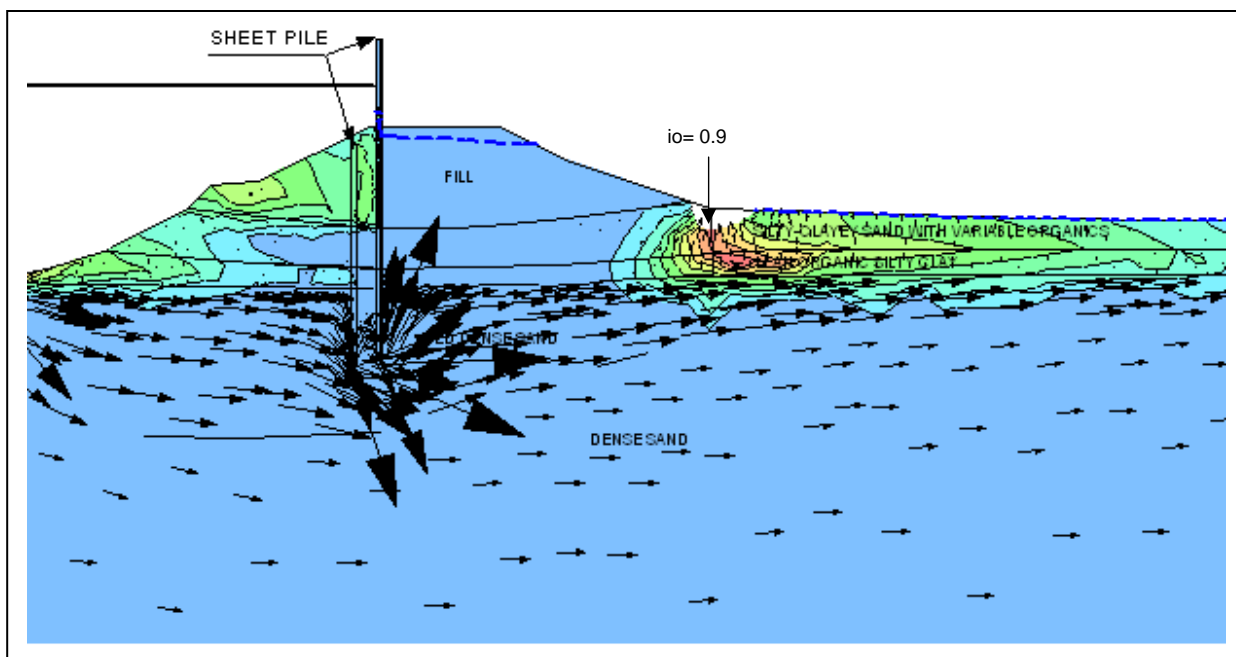


Figure 8.103: Hydraulic gradients for storm surge at +9ft (MSL) in the London Avenue Canal south breach; development of erosion at levee toe. (Maximum gradient $i_o = 0.9$).

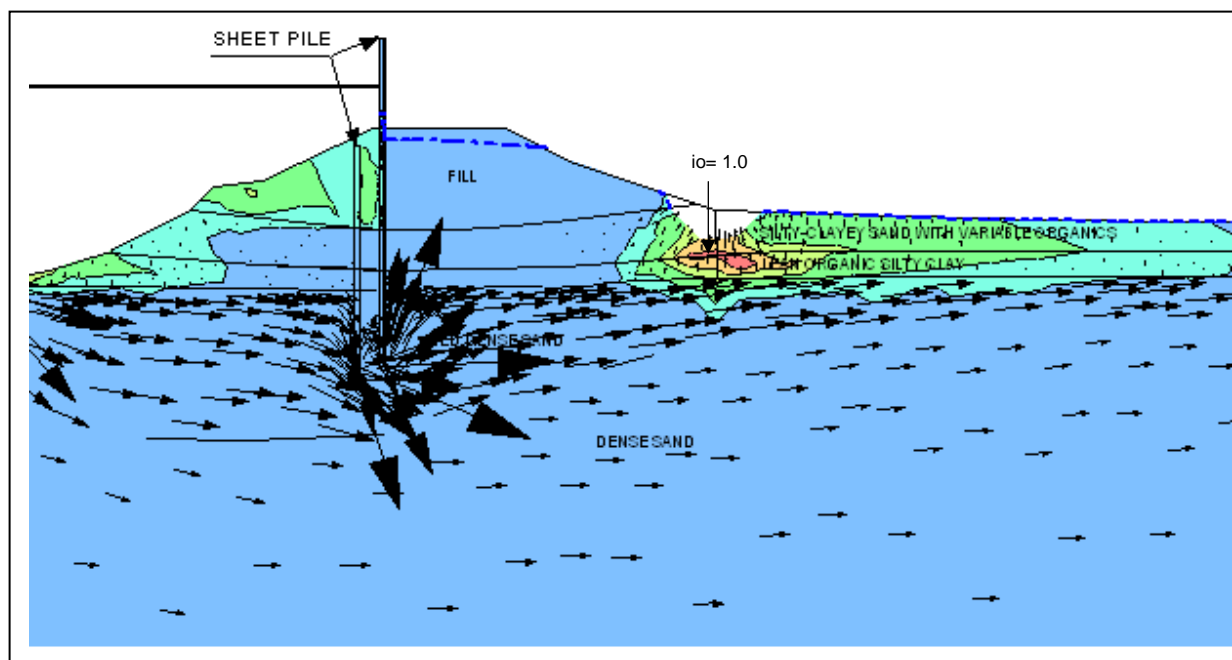


Figure 8.104: Hydraulic gradients for storm surge at +9ft (MSL) in the London Avenue Canal south breach; development of erosion at levee toe. (Maximum gradient $i_o = 1.0$).

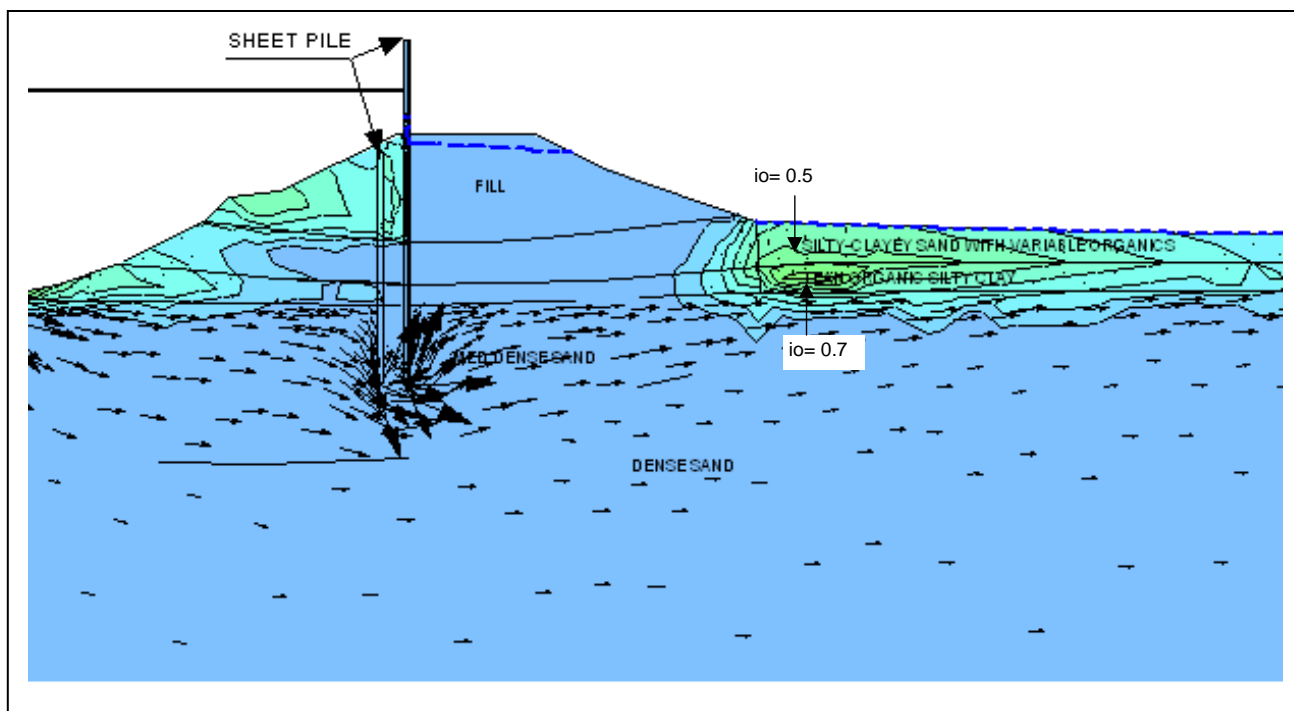


Figure 8.101: Hydraulic gradients for storm surge at +9ft (MSL) in the London Avenue Canal south breach. (Maximum toe exit gradient $i_o = 0.7$)

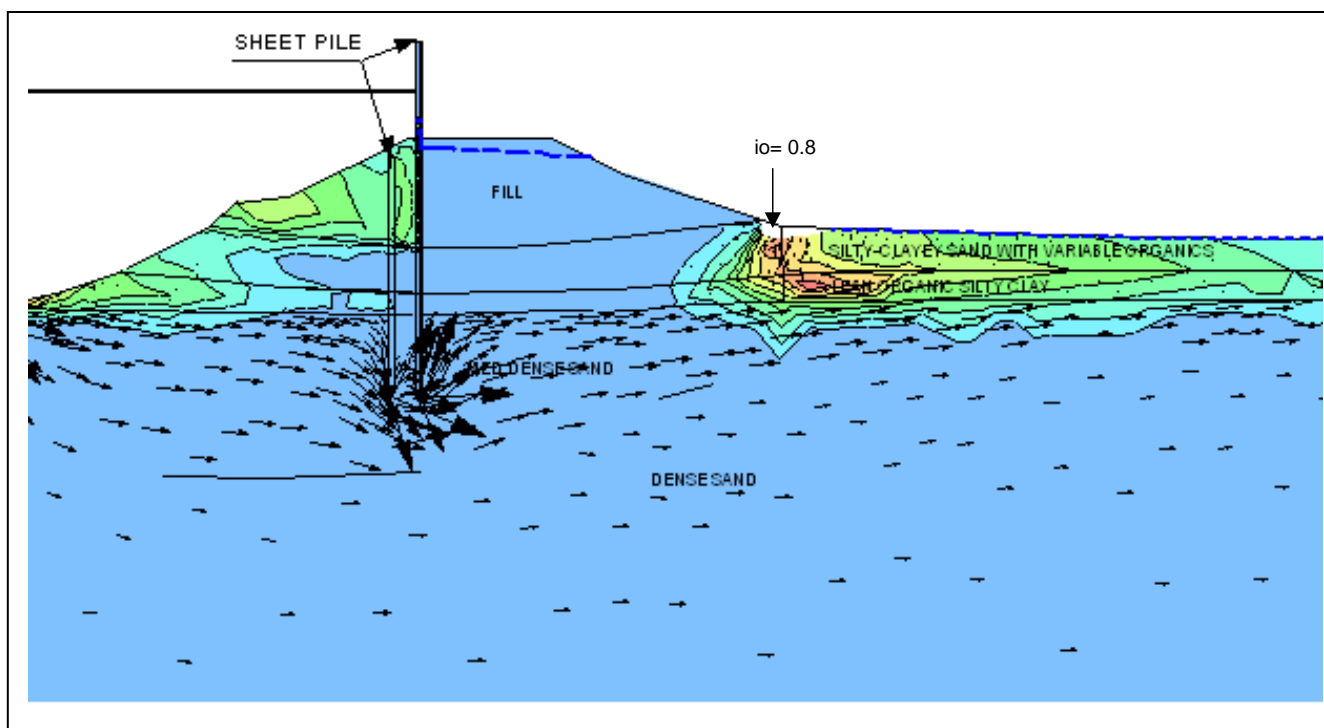


Figure 8.102: Hydraulic gradients for storm surge at +9ft (MSL) in the London Avenue Canal south breach: initiation of erosion at levee toe. (Maximum gradient $i_o = 0.8$)

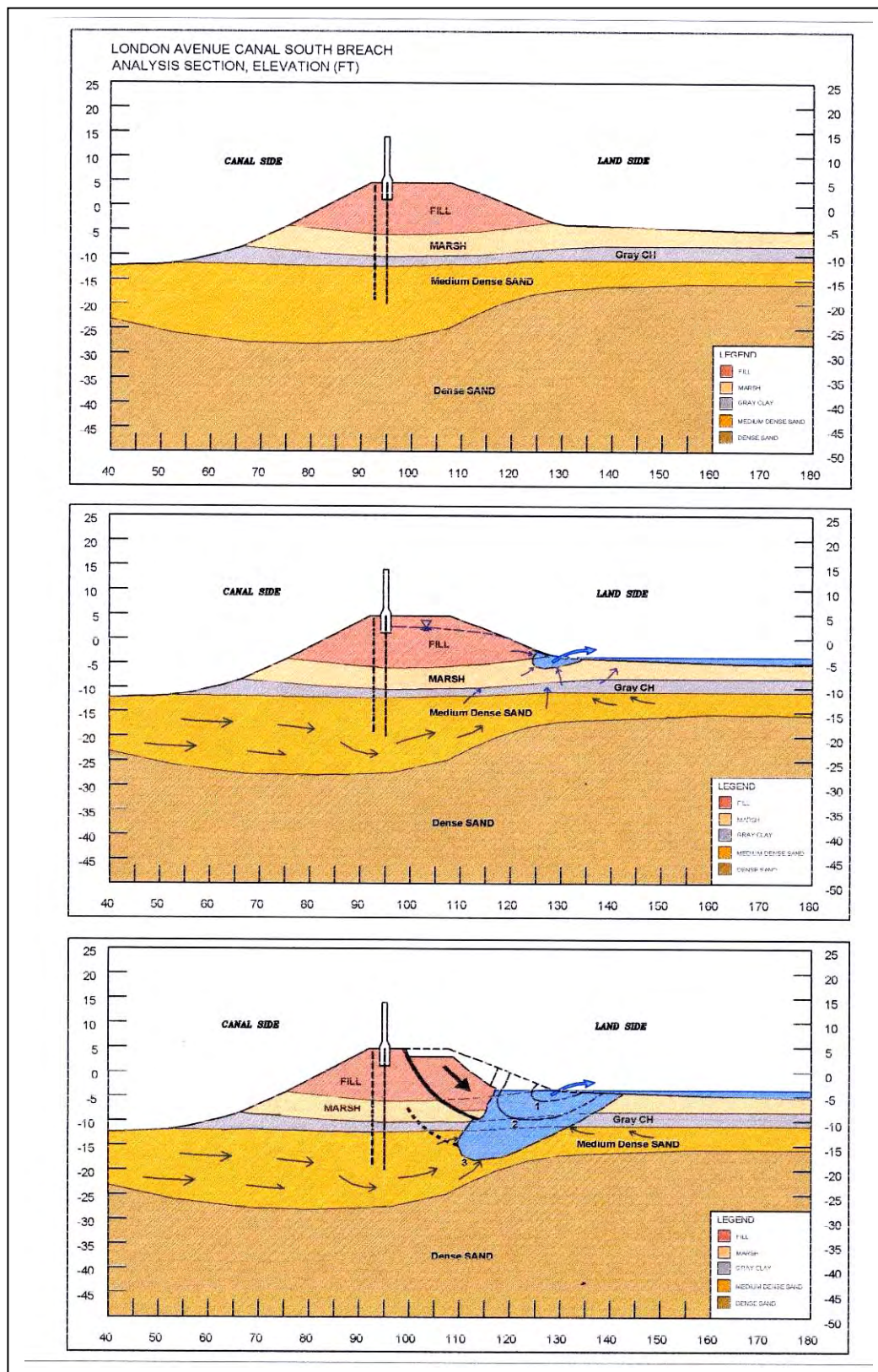


Figure 8.107: Schematic illustration of progressive erosion development at inboard toe.

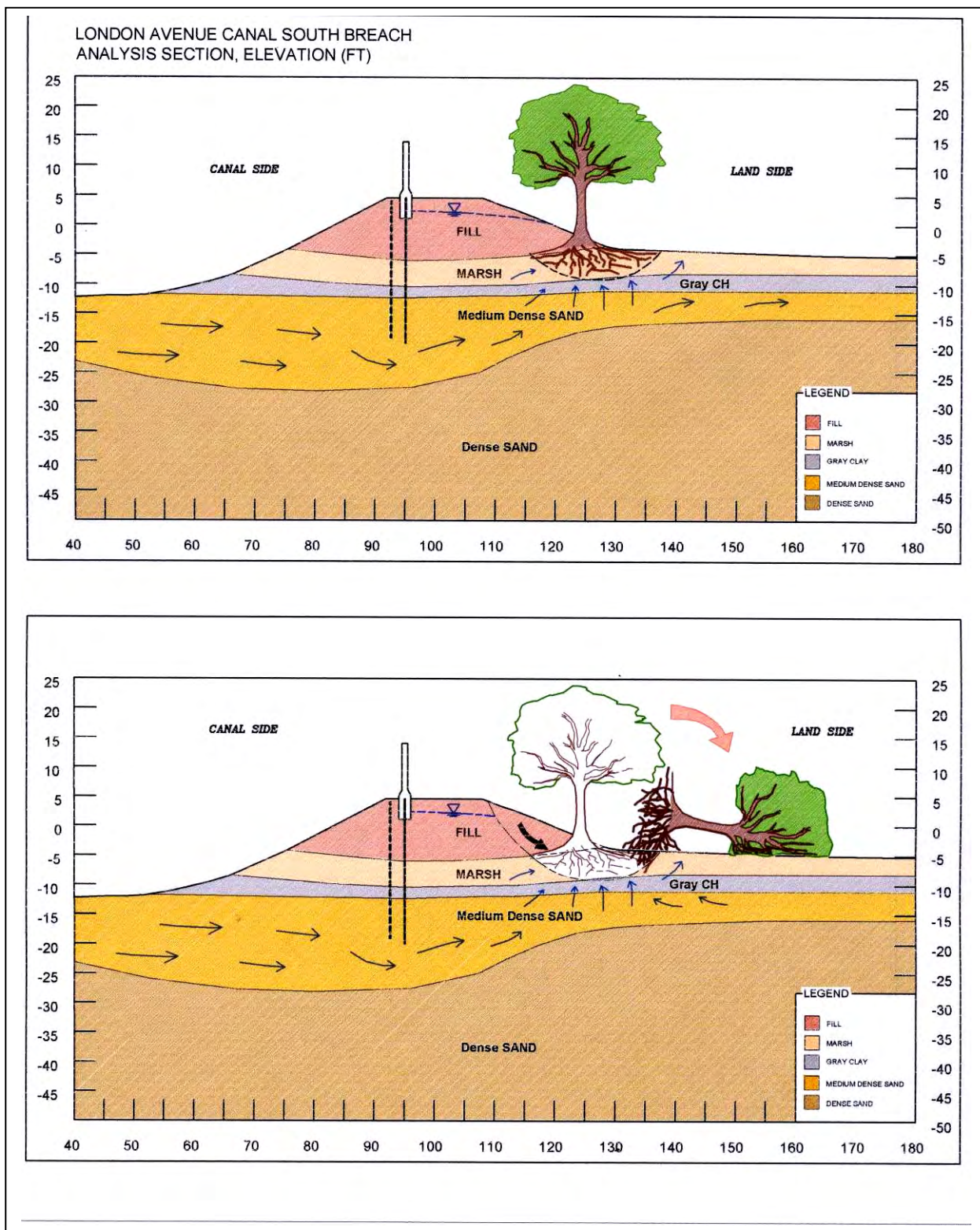


Figure 8.108: Schematic illustration of toppling of tree at inboard toe of levee.



Figure 8.109: View of the breach near the south end of the London Avenue Canal from the canal side in late September of 2005.



Figure 8.109(a): Sample across transition zone at London Avenue Canal, South Breach Section



Figure 8.110: Overhead view of the breach section on the west bank near the north end of the London Avenue Canal.



Figure 8.111: Oblique aerial view of the breached section on the west bank of the London Avenue Canal (North) and the “distressed” section on the east bank (on the left in this photo, which is taken looking to the south.)



Figure 8.112: View of the west bank breach near the north end of the London Avenue Canal from the south end showing the outboard side embankment section still in place.

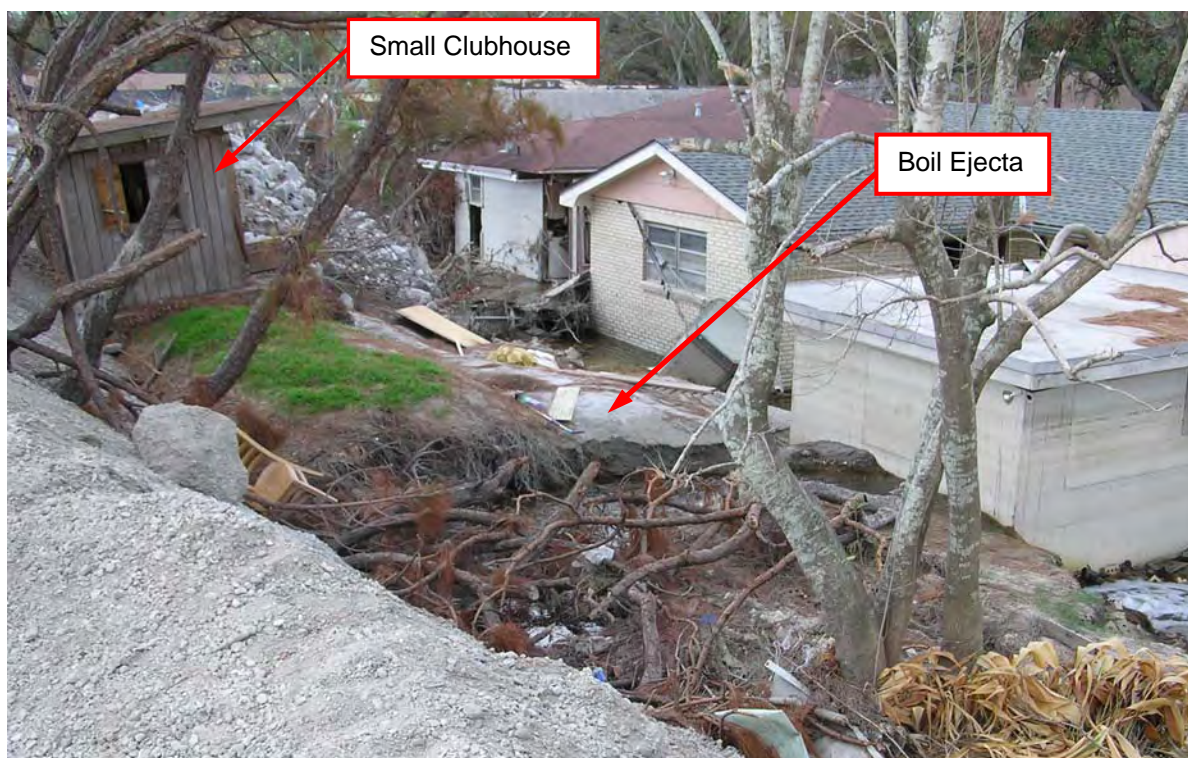


Figure 8.113: View of the inboard of the displaced embankment on the west side of the London Avenue canal showing heaving at the toe (and beneath the small wooden clubhouse), and the boil ejecta in front of the heave feature.



Figure 8.114: Water-filled gap at outboard side of east bank distressed section.

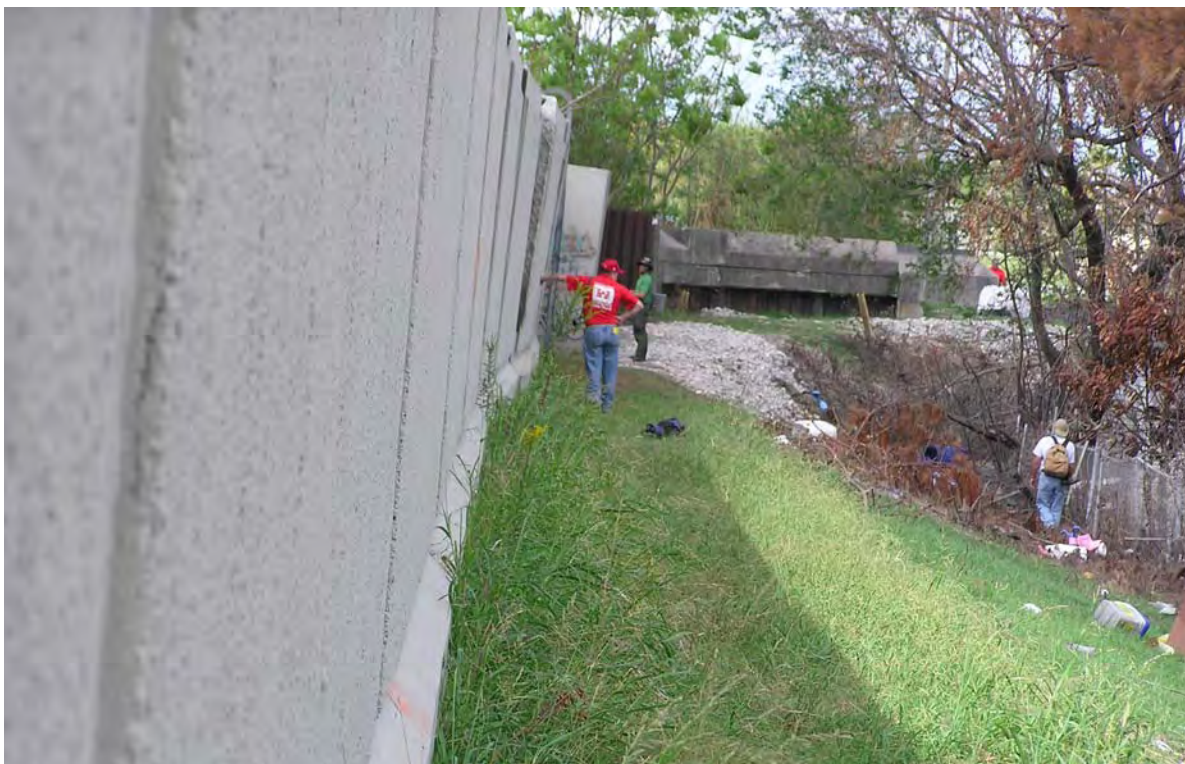


Figure 8.115: Inboard leaning floodwalls on the other side of the floodwalls shown above in Figure 8.112.



Figure 8.116: Sinkholes along the contact at the inboard side base of the floodwall and the levee crest.



Figure 8.117: Sand ejecta from toe boil, and edge of toe thrust feature; London Avenue canal (north) east bank distressed section.

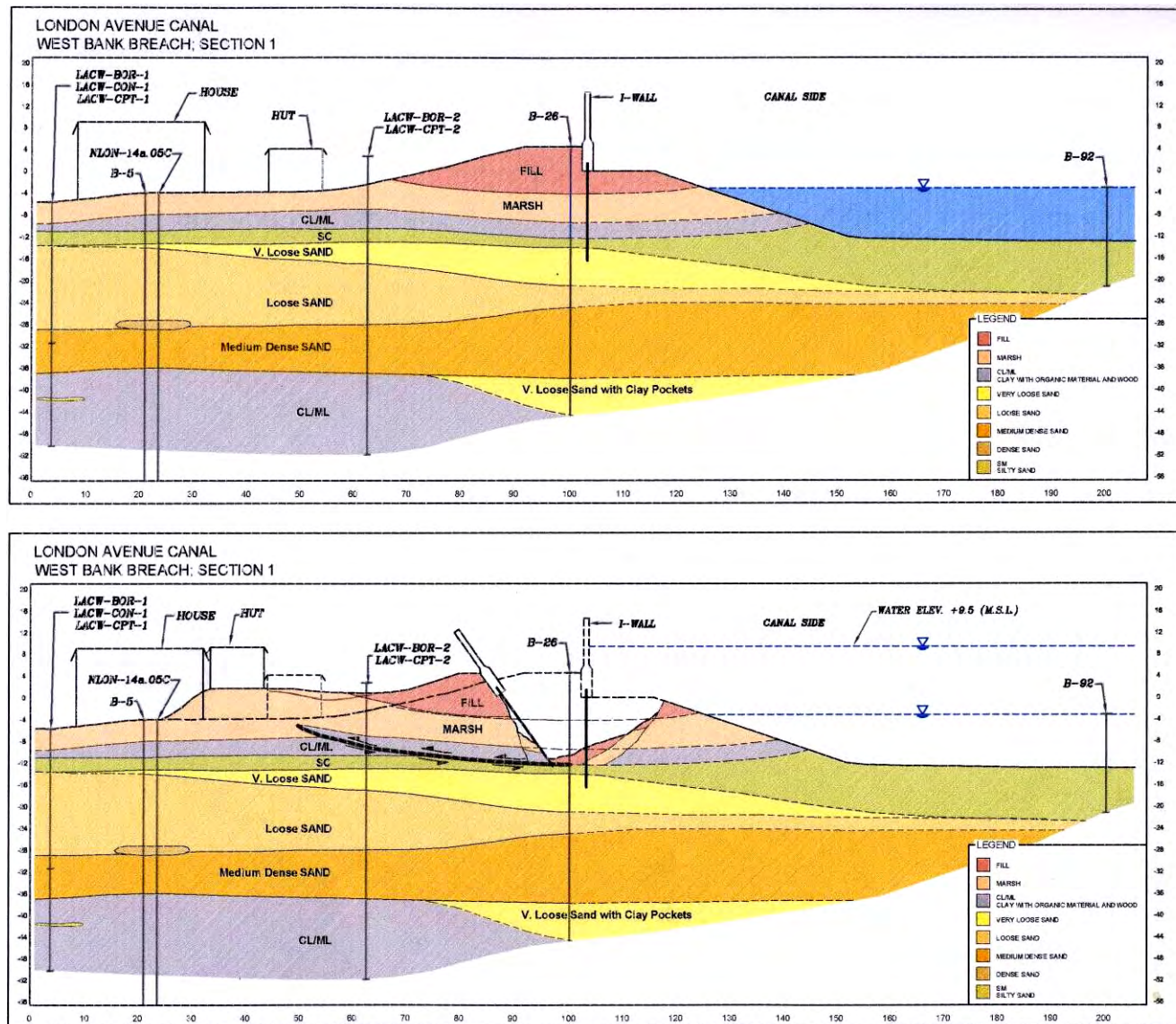


Figure 8.118: Pre-Katrina and Post-Katrina cross-section for London Avenue Canal North, west bank breach section.

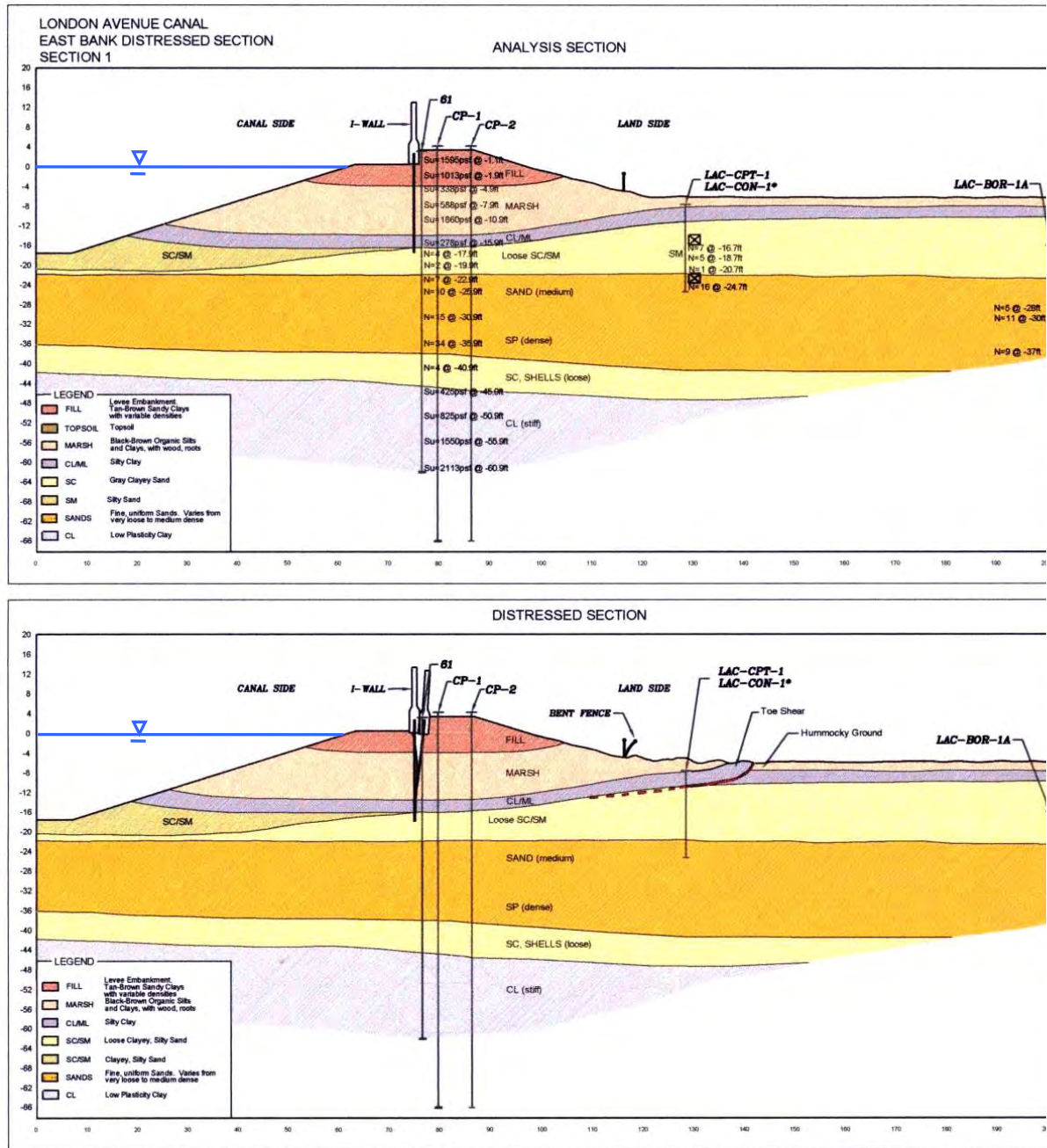
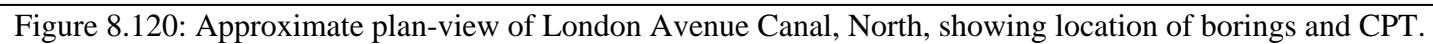


Figure 8.119: Pre-Katrina and Post-Katrina cross-section for London Avenue Canal North, east bank, distressed section.



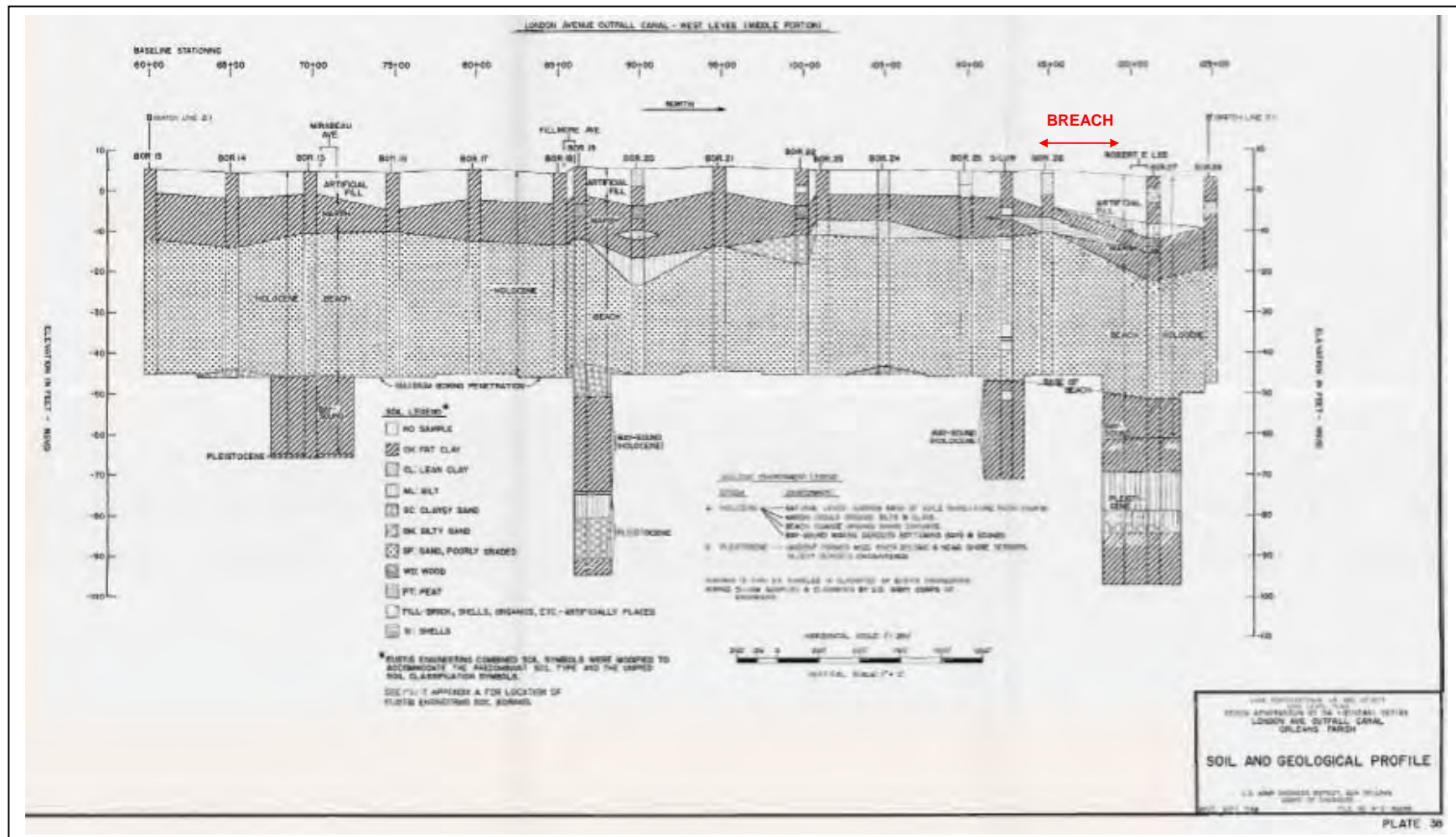
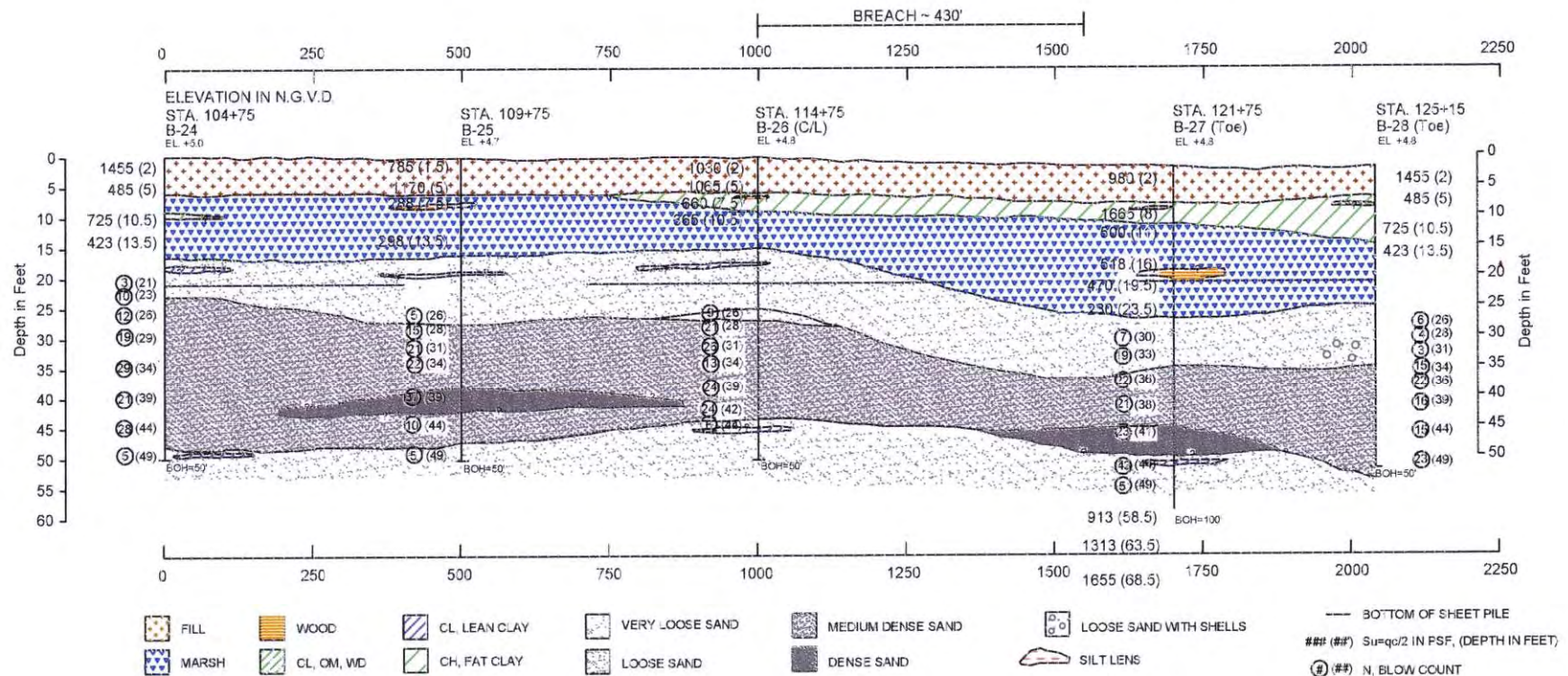


Figure 8.121: Initial longitudinal subsurface profile used for initial design at the London Avenue Canal North, west breach site.
[USACE, DM-19A, Vol. 1, 1989].



London Canal West Levee- Soil Profile
New Orleans, Louisiana

Figure 8.122: Re-interpreted longitudinal subsurface soil profile, showing location of breach section on the west bank of the London Avenue Canal, North.

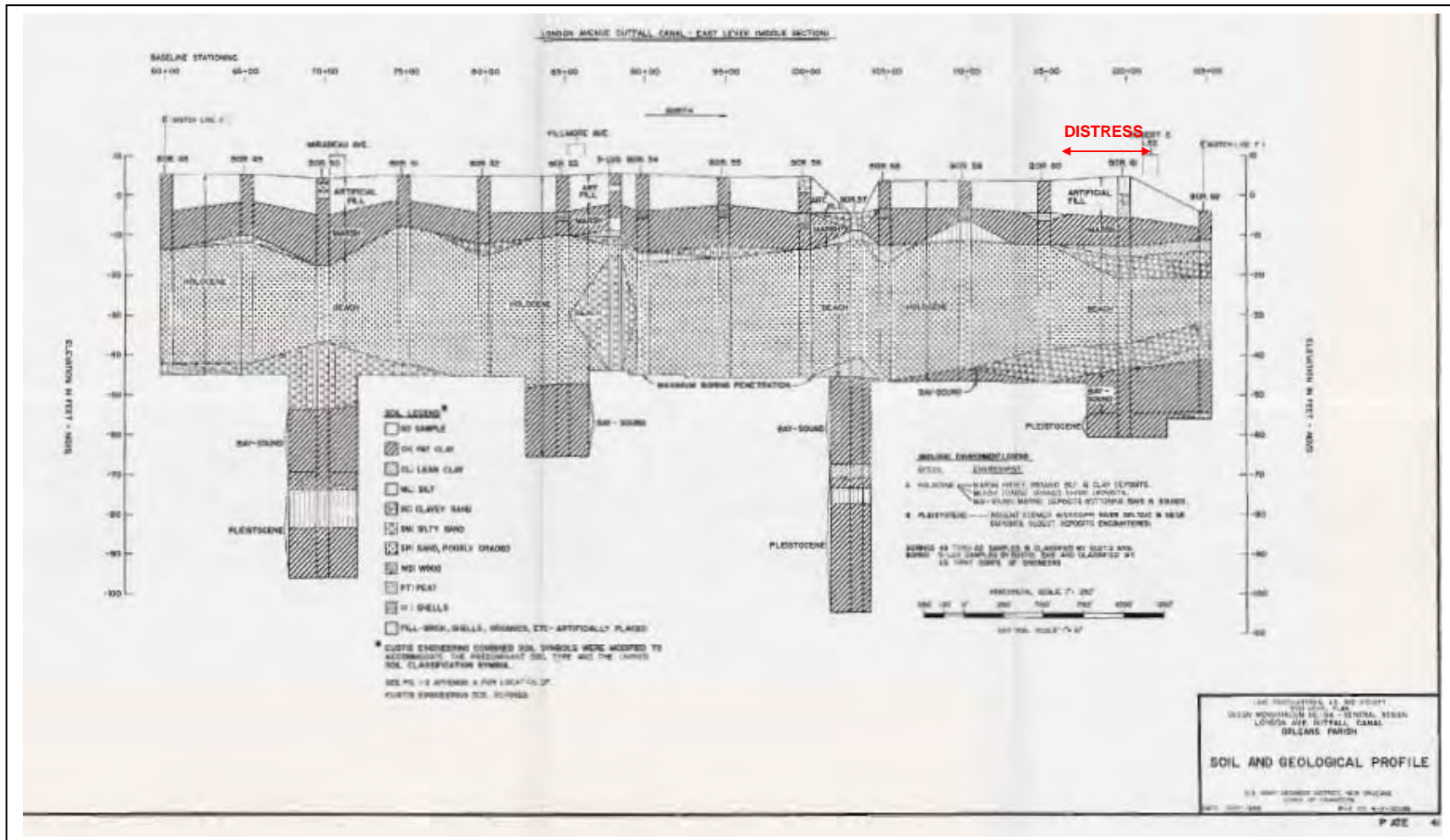


Figure 8.123: Initial longitudinal subsurface profile used for initial design at the London Avenue Canal North, east bank distressed site. [USACE, DM-19A, Vol. 1, 1989].

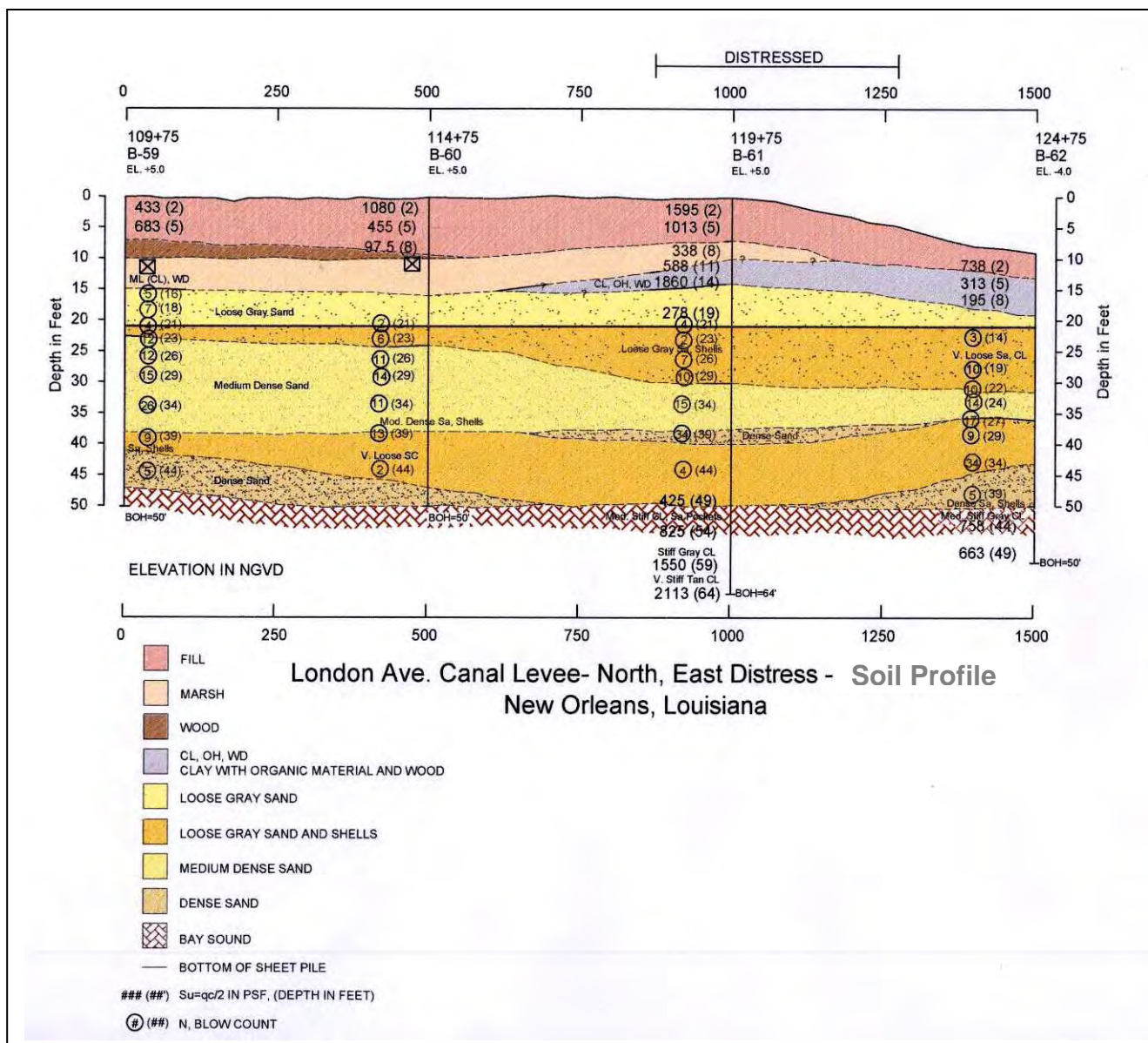


Figure 8.124: Re-interpreted longitudinal subsurface soil profile, showing location of distressed section on the east bank of the London Avenue Canal.

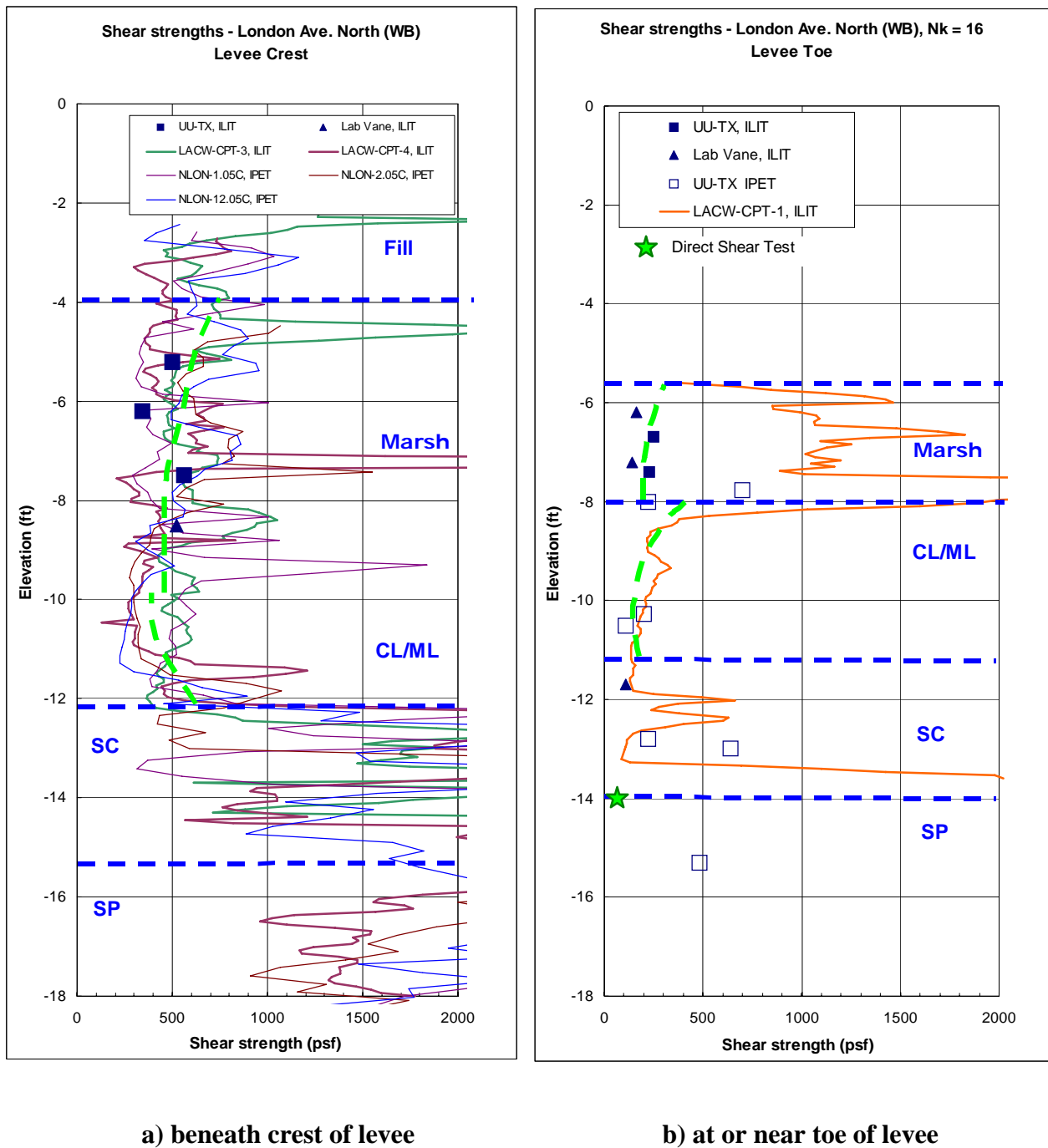


Figure 8.125: Best-estimate for shear strength (S_u) from CPT-data for London Avenue Canal west bank breach section (a) beneath the crest of the levee and (b) at or near the toe of the levee.

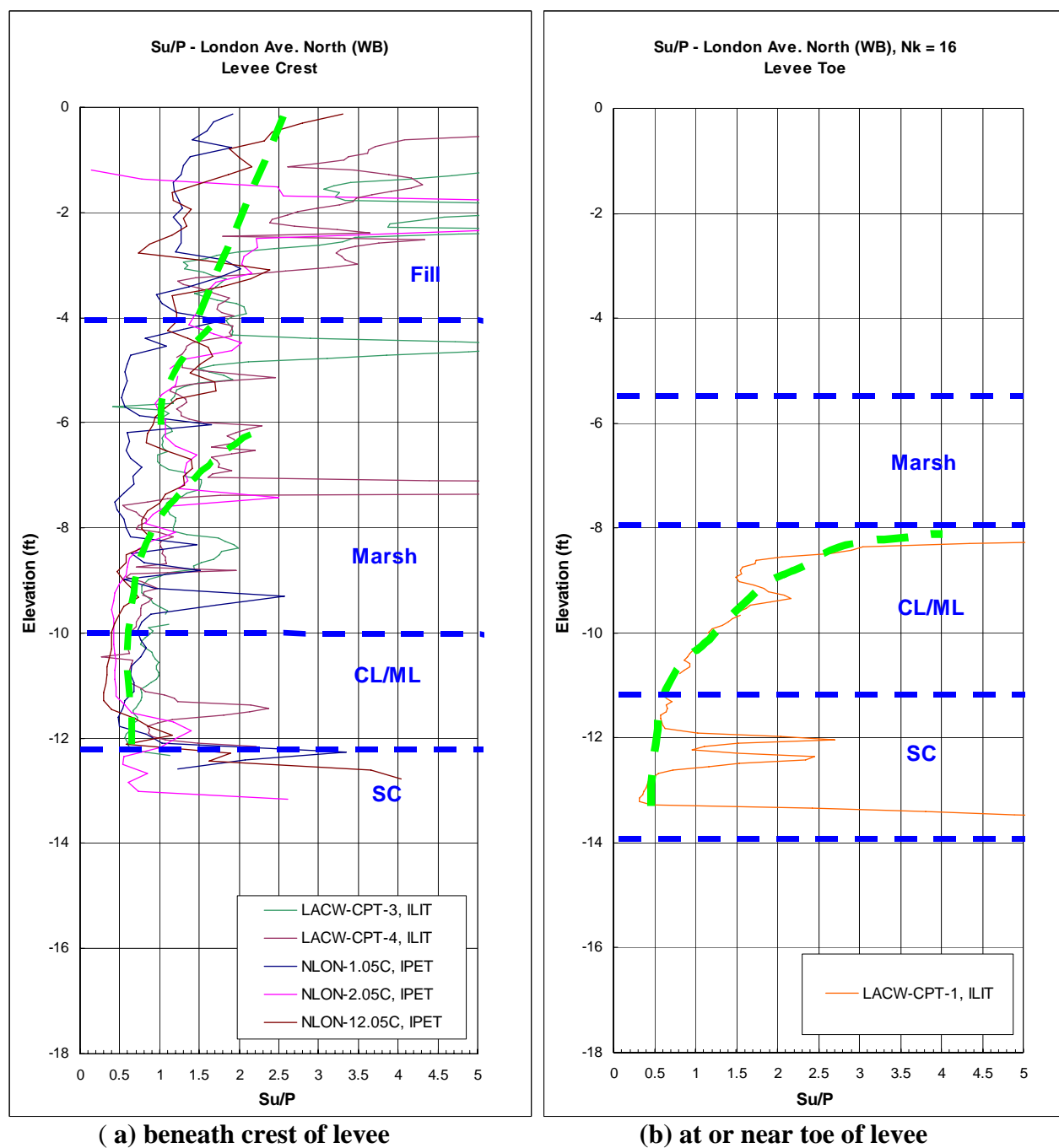


Figure 8.126: Best-estimate for Su/P from CPT-data for London Avenue Canal west bank breach section a) beneath the crest of the levee and b) at or near the toe of the levee.

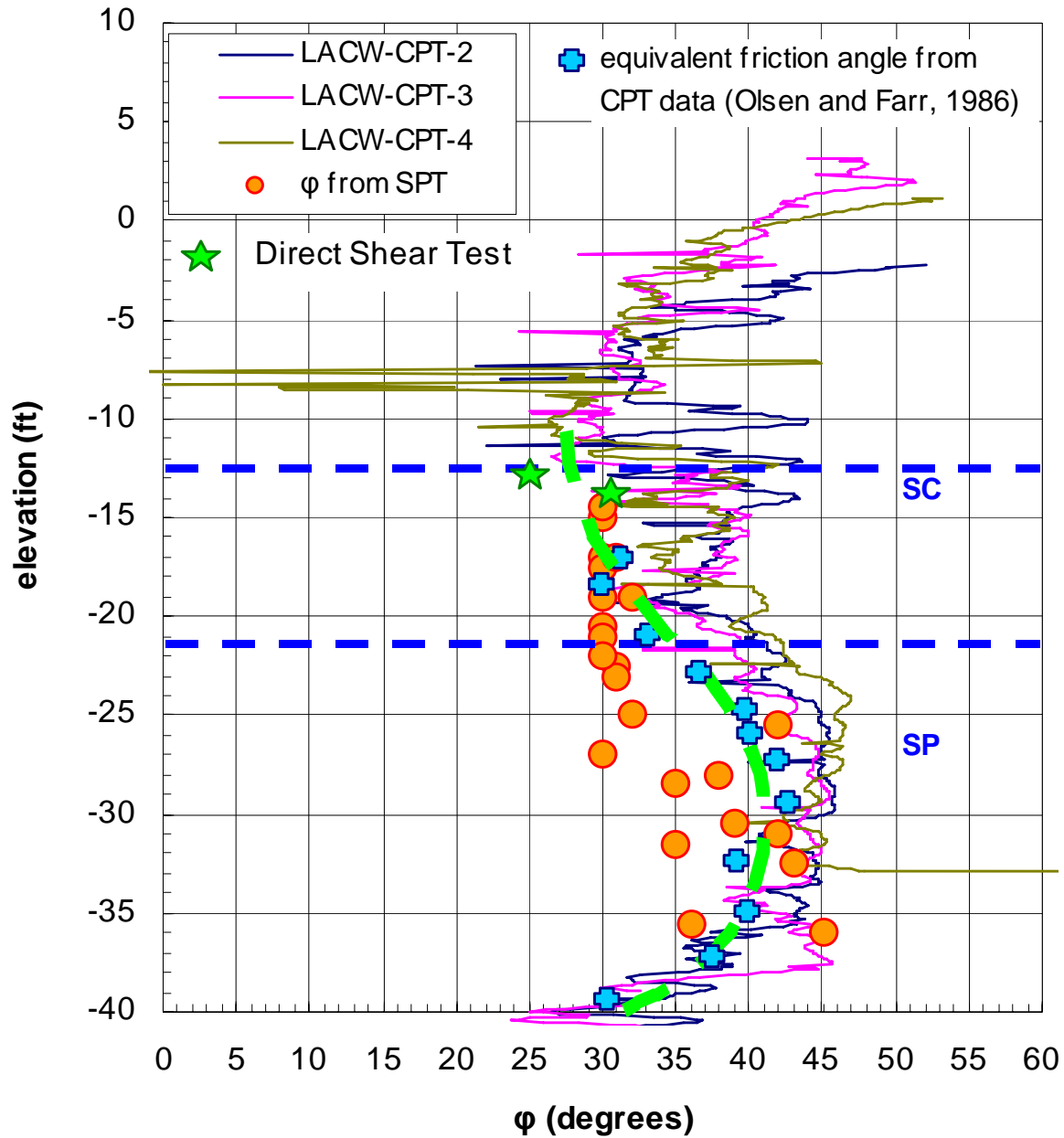


Figure 8.127: Estimation of friction angle for cohesionless materials for London Avenue Canal west bank breach section.

- Notes: a) The continuous lines are from Robertson and Campanella (1983)
 b) Friction angle from SPT is based on R.B. Seed's table
 c) The direct shear test was performed at the UC Berkeley geotechnical laboratory

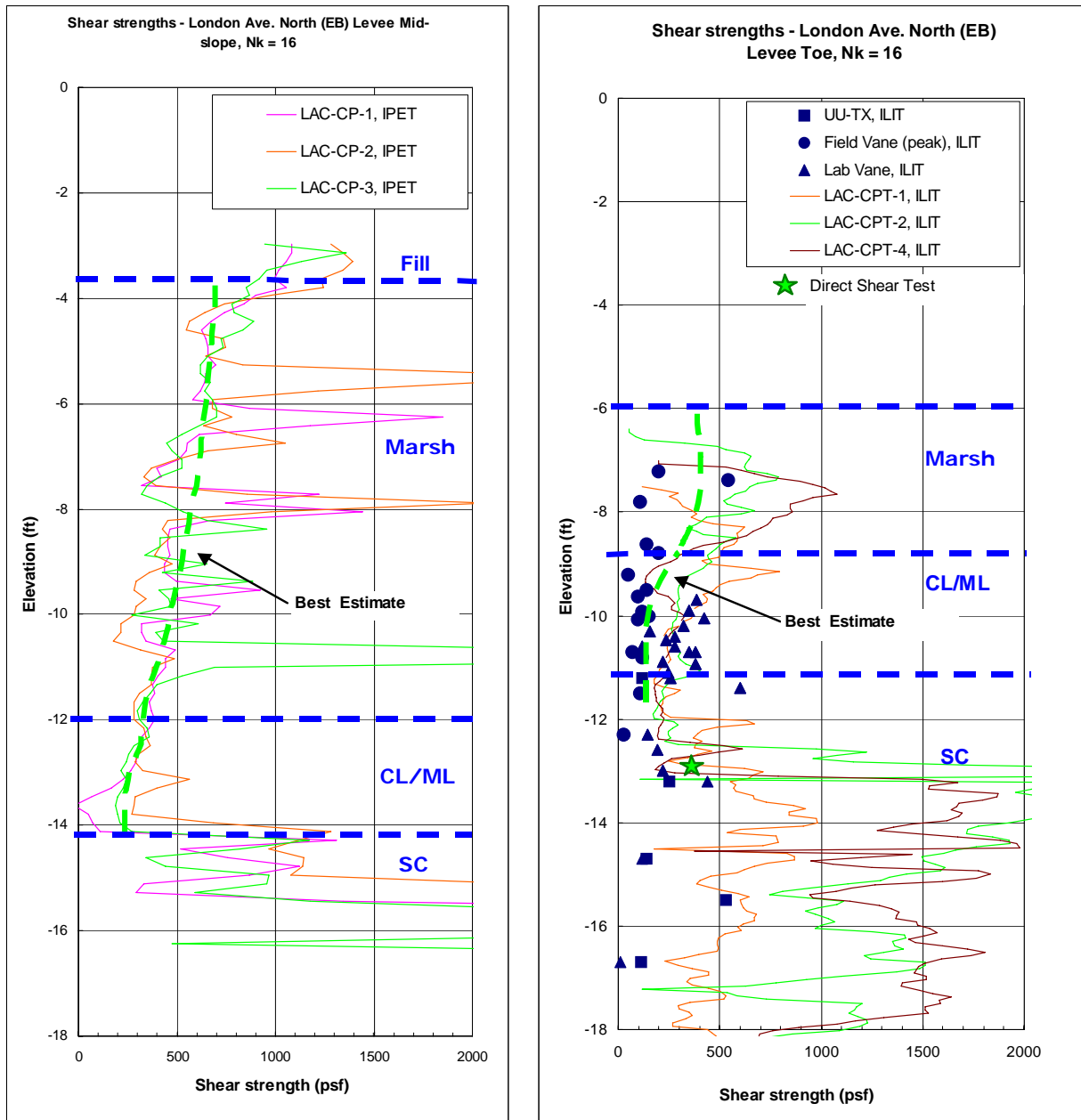


Figure 8.128: Best-estimate for shear strength (S_u) from CPT-data for London Avenue Canal distressed section (east bank) a) beneath the crest of the levee and b) at or near the toe of the levee.

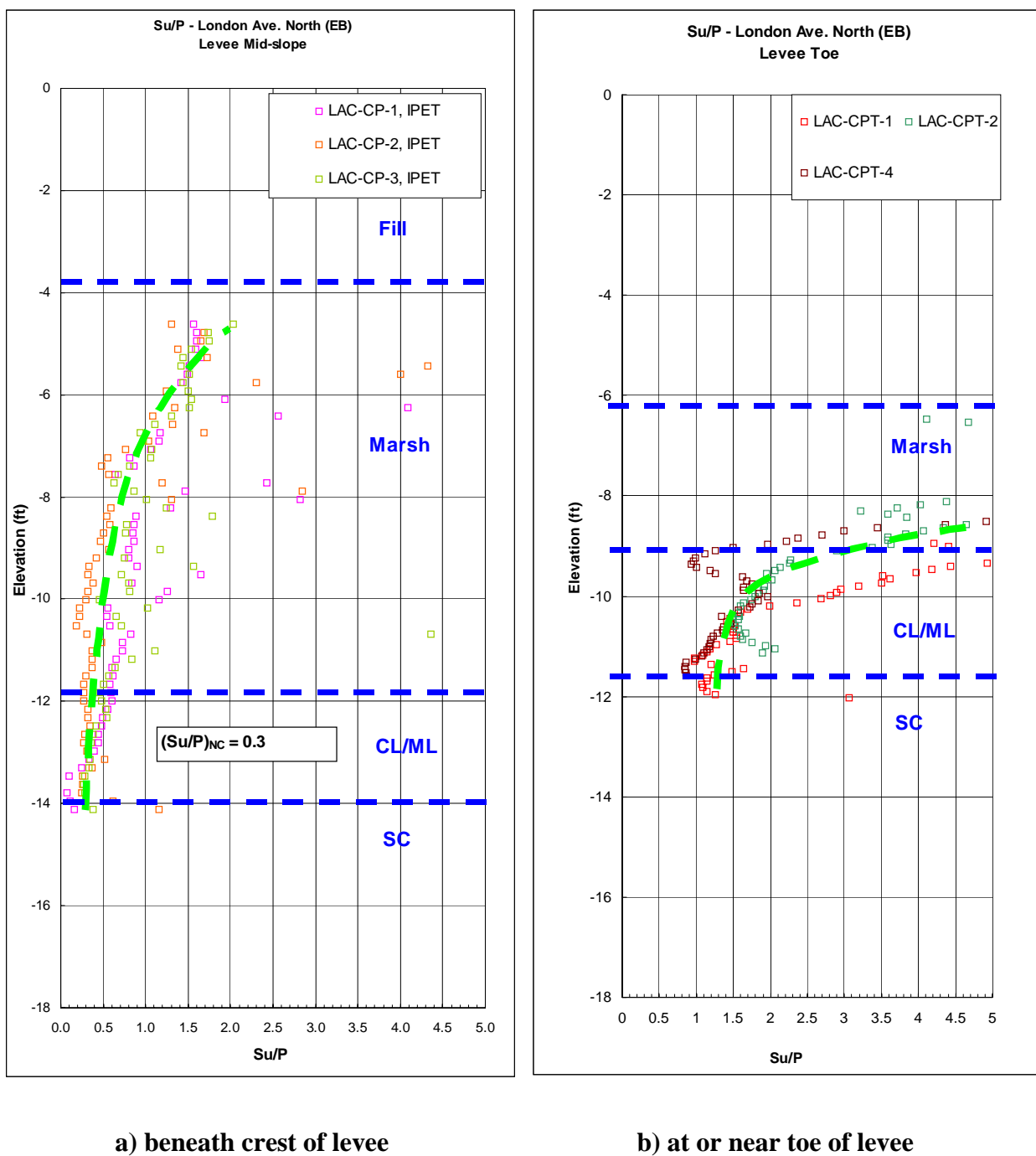


Figure 8.129: Best-estimate for Su/P from CPT-data for London Avenue Canal distressed section (east bank) a) beneath the crest of the levee and b) at or near the toe of the levee

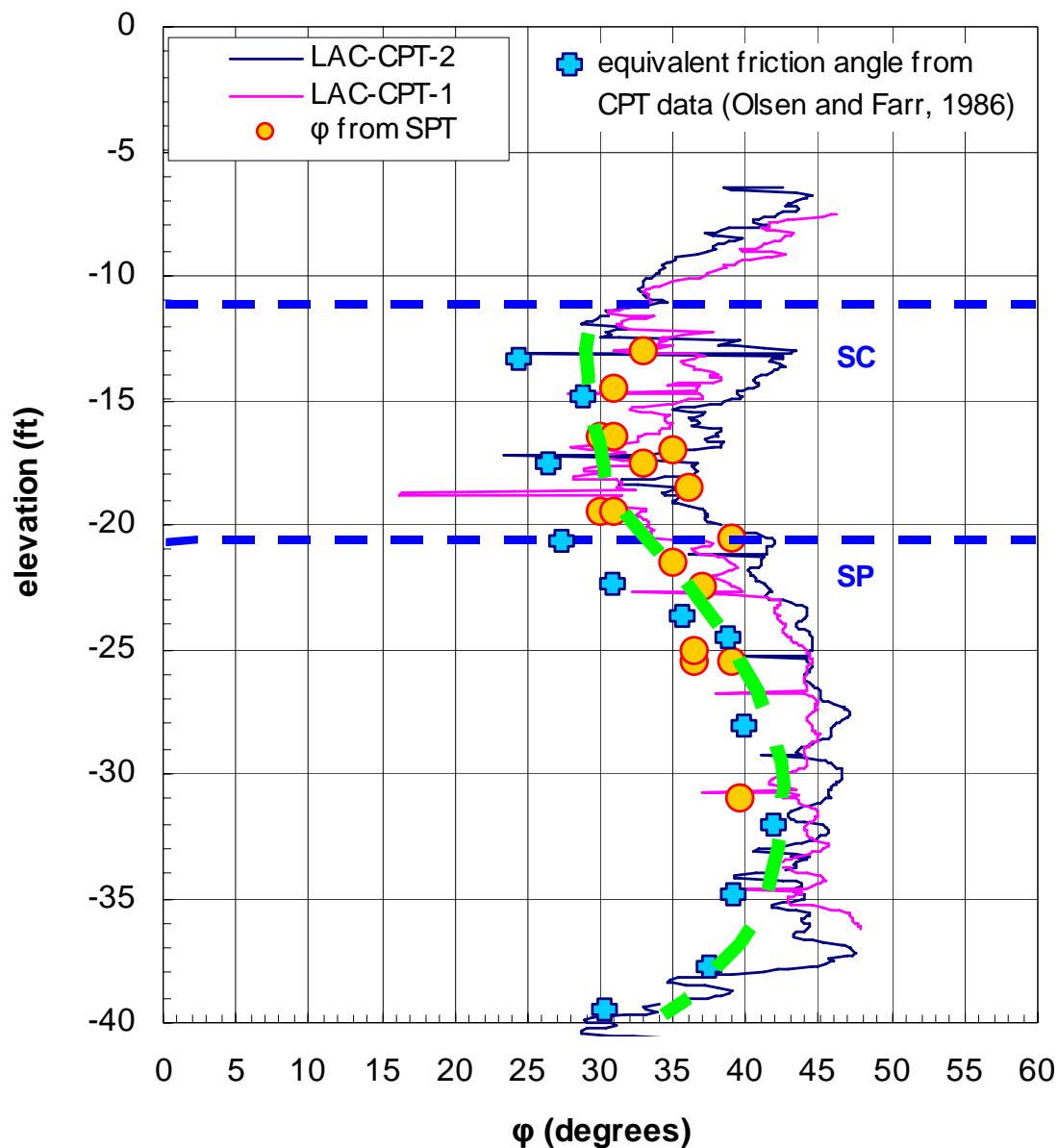


Figure 8.130: Estimation of friction angle for cohesionless materials for London Avenue Canal distressed section (east bank).

Notes: a) The continuous lines are from Robertson and Campanella (1983)
b) Friction angle from SPT is based on R.B. Seed's table

London Avenue Canal, North, East Bank

Geotechnical parameters for the Finite Element Analysis Model

ID	Soil Model	Description	Type	γ_{unsat} [lb/ft ³]	γ_{sat} [lb/ft ³]	k_x [ft/day]	k_y [ft/day]	ν [-]	E_{ref} [lb/ft ²]	c_{ref} [lb/ft ²]	ϕ [°]
fill	Mohr Coulomb	(Non-engineered Fill / Fill)	Undrained	70	90	2.80E-03	2.80E-04	0.35	1.00E+06	800	0.001
marsh		marsh	Undrained	50	80	28.3	2.83	0.35	2.80E+05	550	0.001
CL/ML		silty low-PI clay (traces organic)	Undrained	70	90	2.80E-03	2.80E-04	0.35	9.00E+05	250	0.001
SC/SM		clayey, silty sand	Drained	90	100	2.33	1.15	0.3	5.00E+05	0.001	30
SP_1		loose sand	Drained	90	100	2.33	2.33	0.25	9.00E+05	0.001	33
SP_2		medium dense sand	Drained	100	110	2.33	2.33	0.25	1.00E+06	0.001	36
SP_3		dense sand	Drained	110	115	2.33	2.33	0.25	1.50E+06	0.001	40
CH		grey clay (Bay Sound)	Undrained	95	105	2.80E-03	2.80E-04	0.35	1.00E+06	600	0.001

London Avenue Canal, North, West Bank

Geotechnical parameters for the Finite Element Analysis Model

ID	Soil Model	Description	Type	γ_{unsat} [lb/ft ³]	γ_{sat} [lb/ft ³]	k_x [ft/day]	k_y [ft/day]	ν [-]	E_{ref} [lb/ft ²]	c_{ref} [lb/ft ²]	ϕ [°]
fill	Mohr Coulomb	(Non-engineered Fill / Fill)	Undrained	70	90	2.80E-03	2.80E-04	0.35	1.00E+06	800	0.001
marsh		marsh	Undrained	50	80	28.3	2.83	0.35	2.80E+05	450	0.001
CL/ML		silty low-PI clay (traces organic)	Undrained	70	90	2.80E-03	2.80E-04	0.35	9.00E+05	300	0.001
SC/SM		clayey, silty sand	Drained	90	100	2.33	1.15	0.3	5.00E+05	0.001	29
SP_1		loose sand	Drained	90	100	2.33	2.33	0.25	9.00E+05	0.001	33
SP_2		medium dense sand	Drained	100	110	2.33	2.33	0.25	1.00E+06	0.001	36
SP_3		dense sand	Drained	110	115	2.33	2.33	0.25	1.50E+06	0.001	40
CH		grey clay (Bay Sound)	Undrained	95	105	2.80E-03	2.80E-04	0.35	1.00E+06	600	0.001

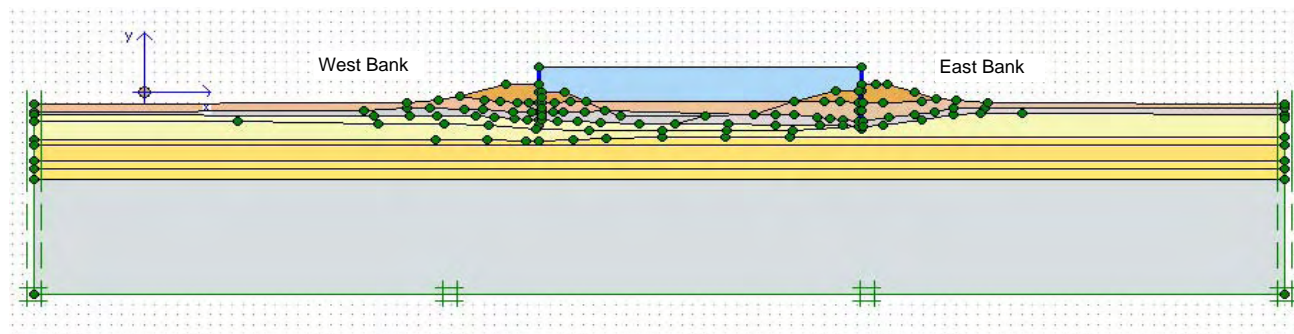


Figure 8.131: Geometry and input parameters for FEM analyses for London Avenue Canal, North (east and west banks)

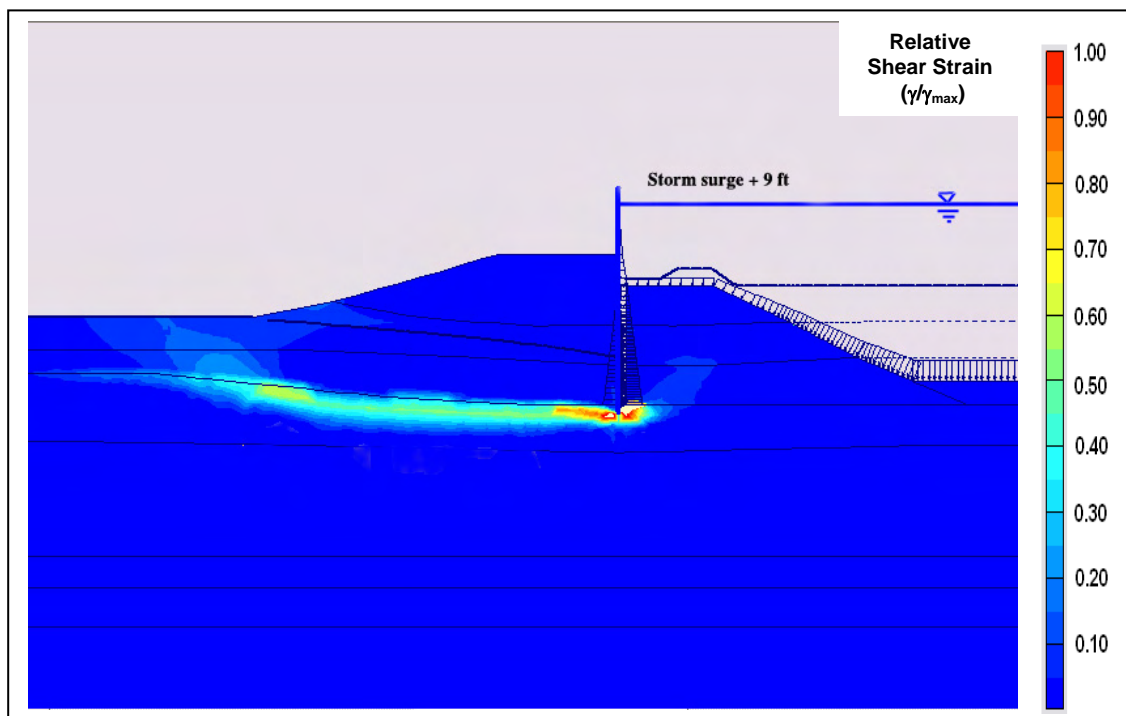


Figure 8.132: Normalized shear strain contours (shear strain divided by strain to failure) for a storm surge at Elev. + 9 feet (MSL) at the London Avenue Canal breach site (west bank); gapping at outboard toe of floodwall is developed to full depth.

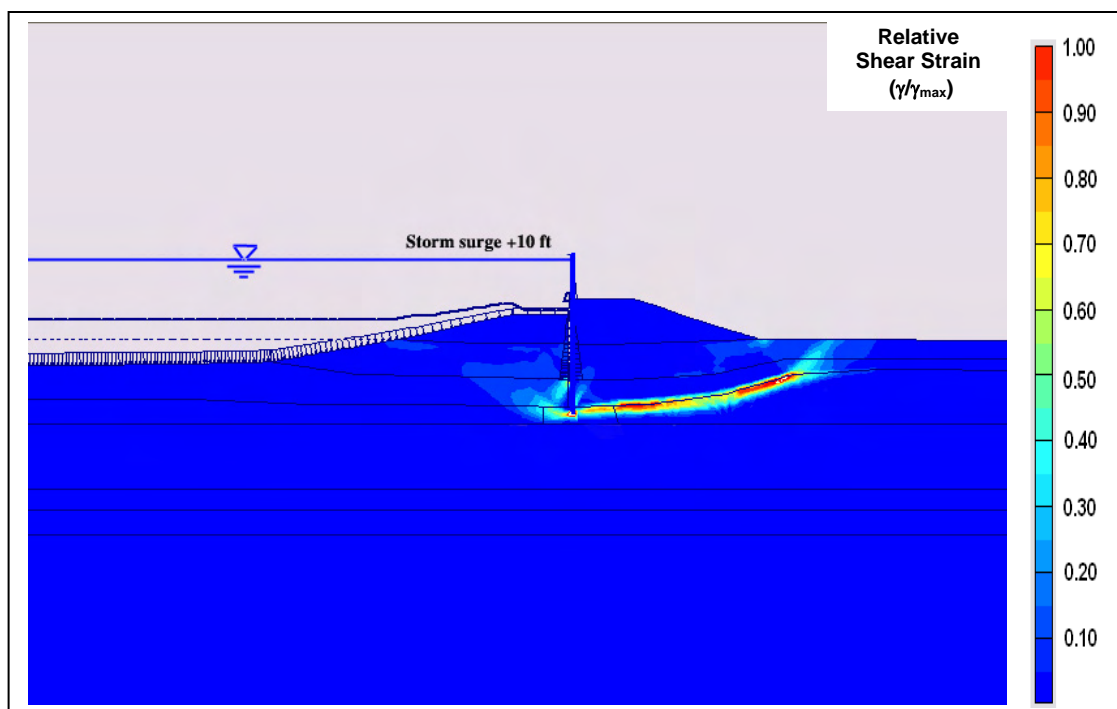


Figure 8.133: Normalized shear strain contours (shear strain divided by strain to failure) for a storm surge at Elev. + 10 feet (MSL) at the London Avenue Canal distressed site (east bank); gapping at outboard toe of floodwall is developed.

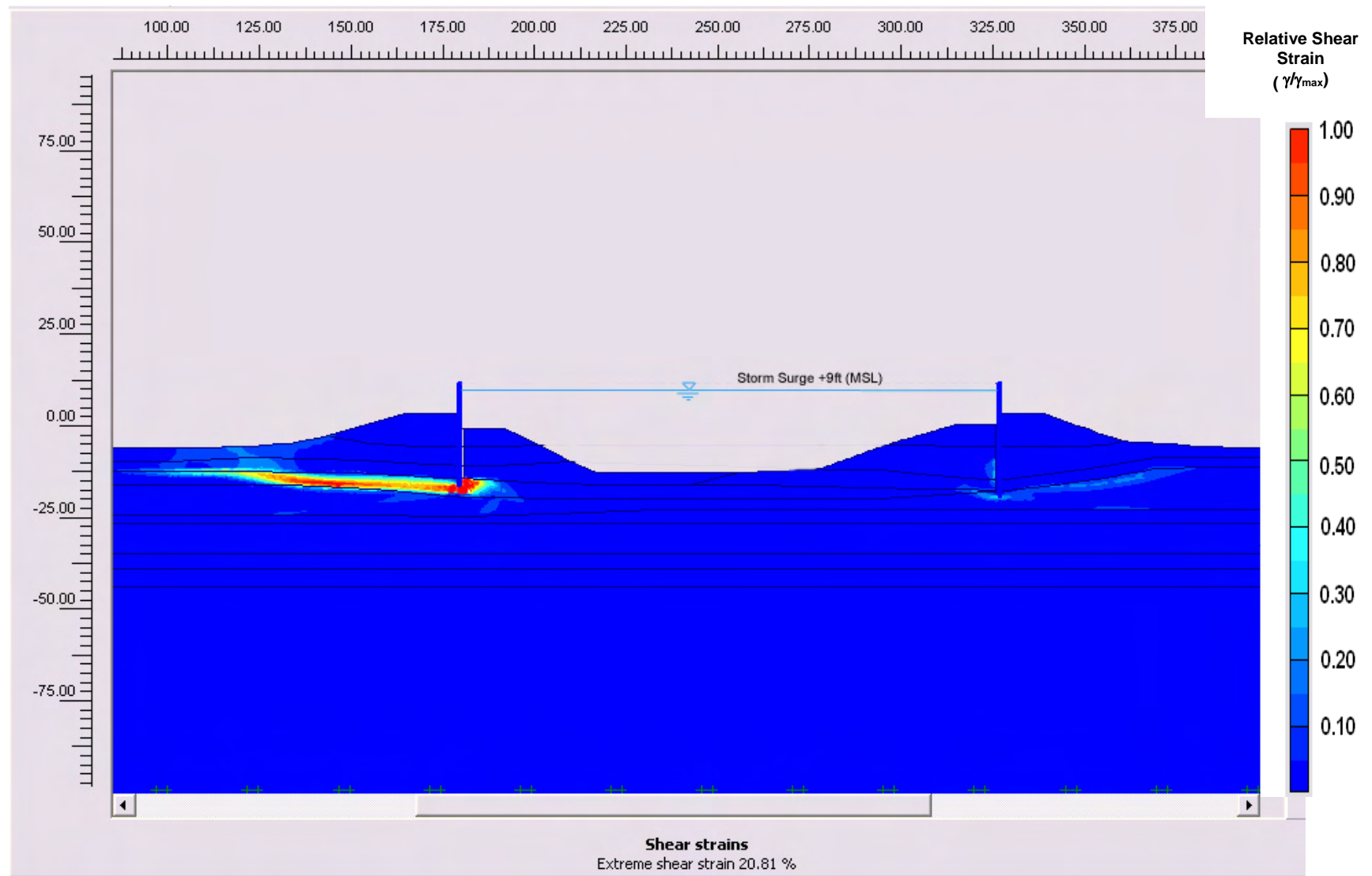


Figure 8.134: Normalized shear strain contours (shear strain divided by strain to failure) for a storm surge at Elev. + 9 feet (MSL) at the

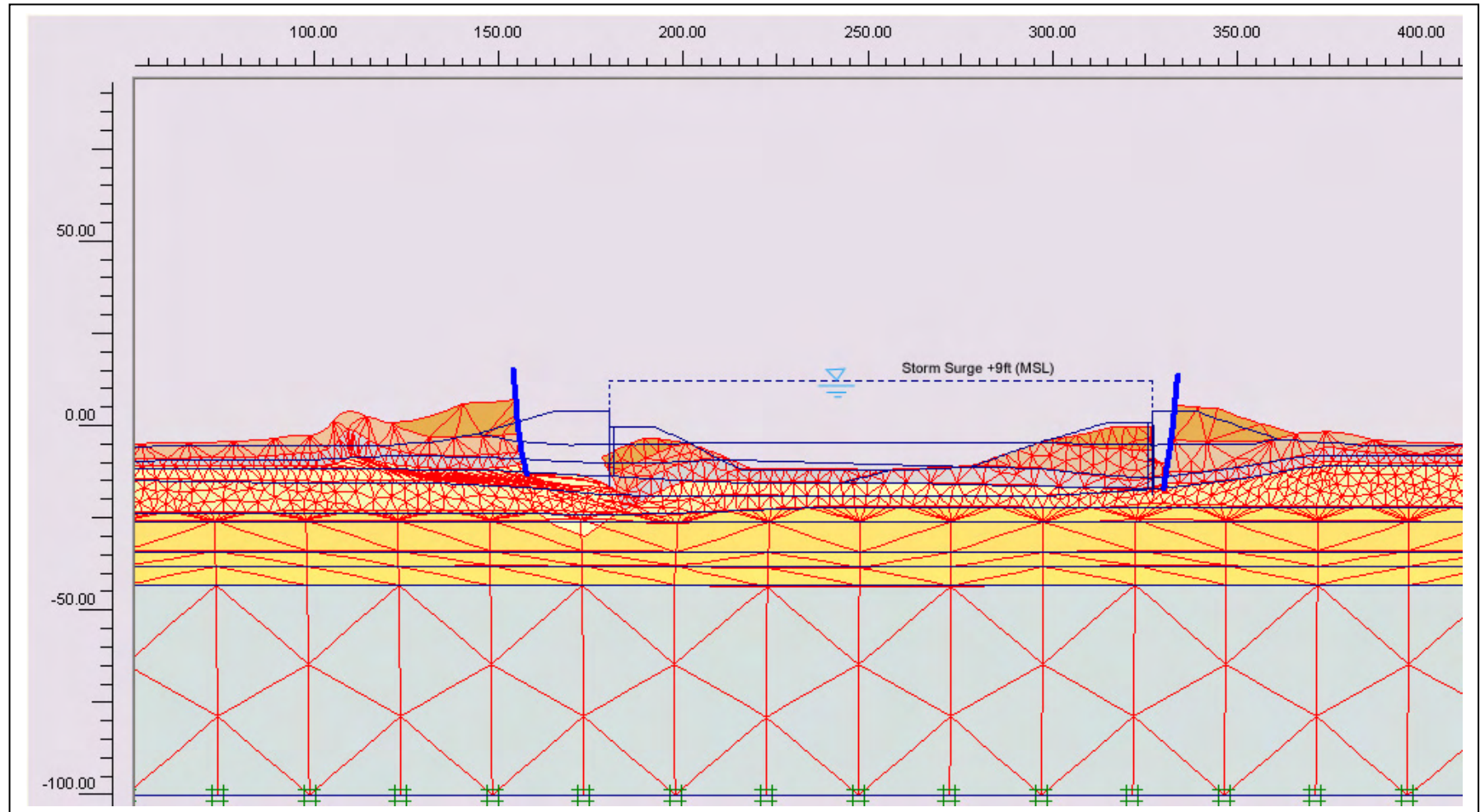


Figure 8.135: Deformed mesh for a storm surge elevation + 9 feet (MSL), London Avenue Canal (east and west banks). Displacements

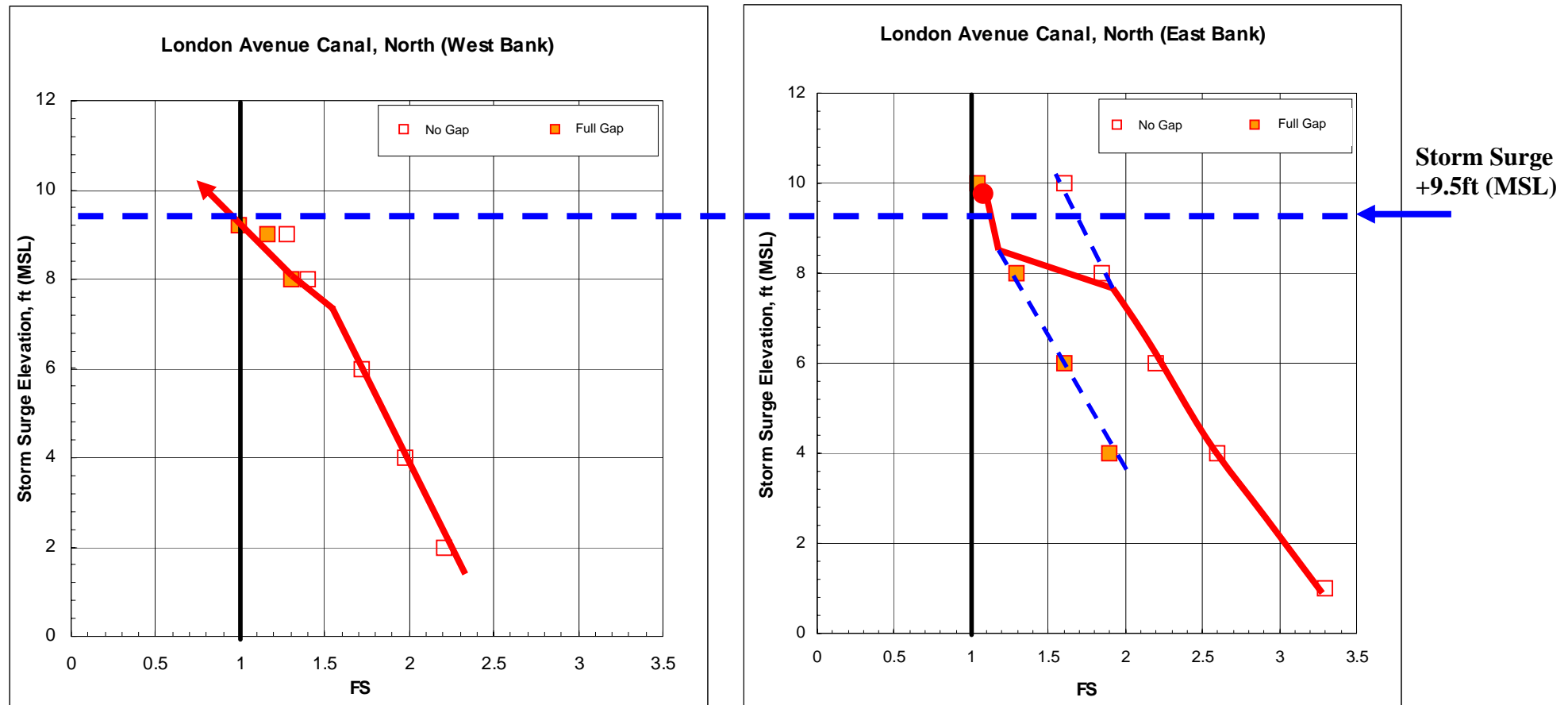


Figure 8.136: Calculated Factors of Safety for two modes based on PLAXIS analyses of the London Avenue Canal breach and distressed section for various canal water elevations; showing the best-estimated path to failure.

ID	Soil Model	Type	γ	c	ϕ	k
			[lb/ft ³]	[lb/ft ²]	[°]	[ft/hr]
Upper fill	Mohr-Coulomb	Undrained	100	800	0	1.17E-04
Marsh, crest	Mohr-Coulomb	Undrained	80	475	0	1.17
Marsh, free field	Mohr-Coulomb	Undrained	80	200	0	1.17
CL/ML, crest	Mohr-Coulomb	Undrained	85	500	0	2.00E-04
CL/ML, free field	Mohr-Coulomb	Undrained	85	300	0	2.00E-04
SC	Mohr-Coulomb	Drained	100	0	30	0.097
Loose Sand	Mohr-Coulomb	Drained	100	0	33	0.333
Dense Sand	Mohr-Coulomb	Drained	105	0	36	0.09
Stiff Clay	Mohr-Coulomb	Undrained	90	600	0	2.00E-04

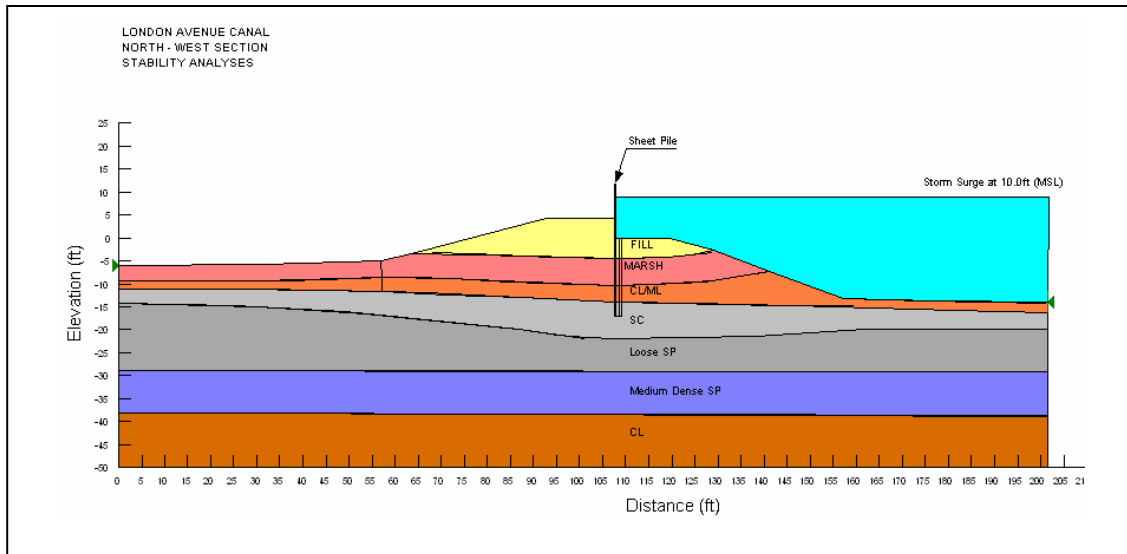


Figure 8.137: Geometry and input parameters for Limit Equilibrium and Steady State seepage analyses for London Avenue Canal North, West bank.

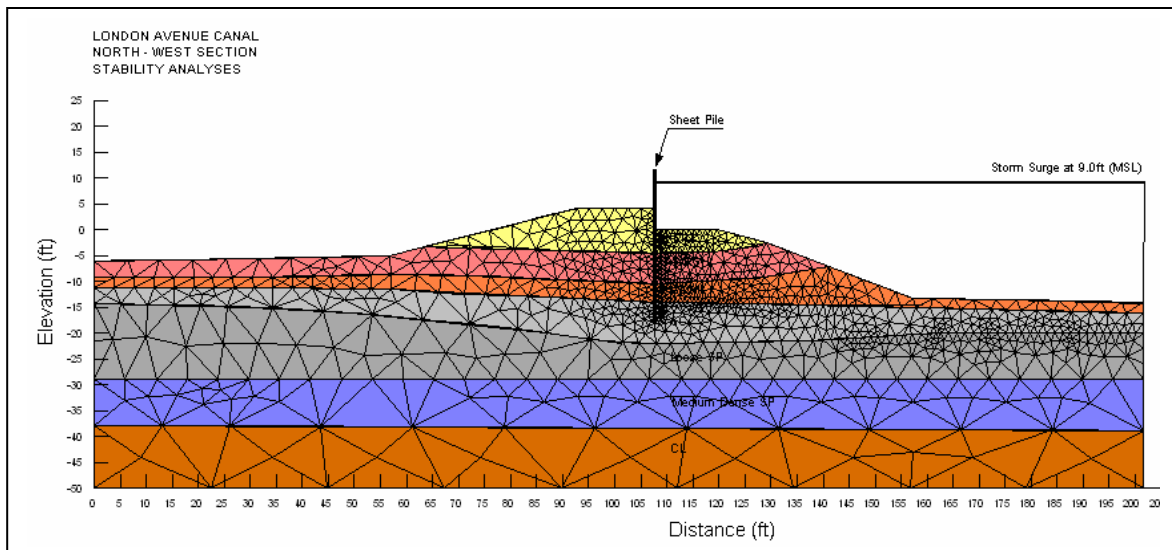


Figure 8.138: Finite Difference mesh for Steady State seepage Analyses for London Avenue Canal North, West bank.

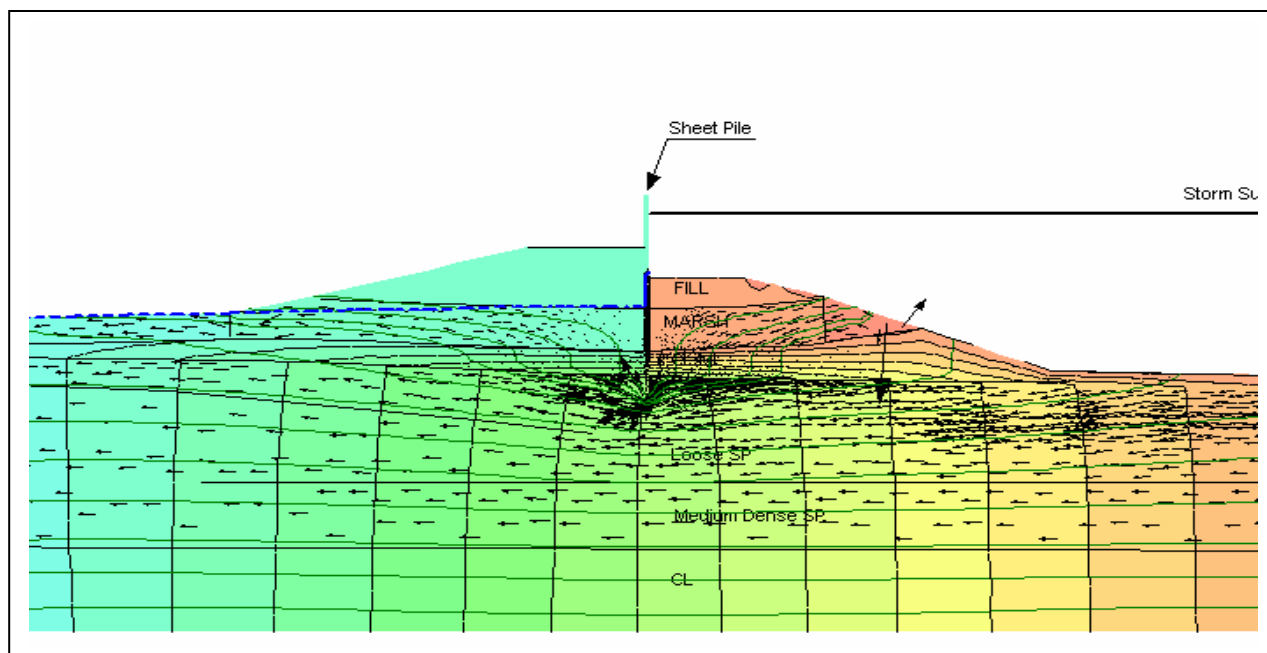


Figure 8.139: Flow net generation without the gapping in the outboard toe of the floodwall, London Avenue Canal North, West bank. Storm surge at 9ft (MSL).

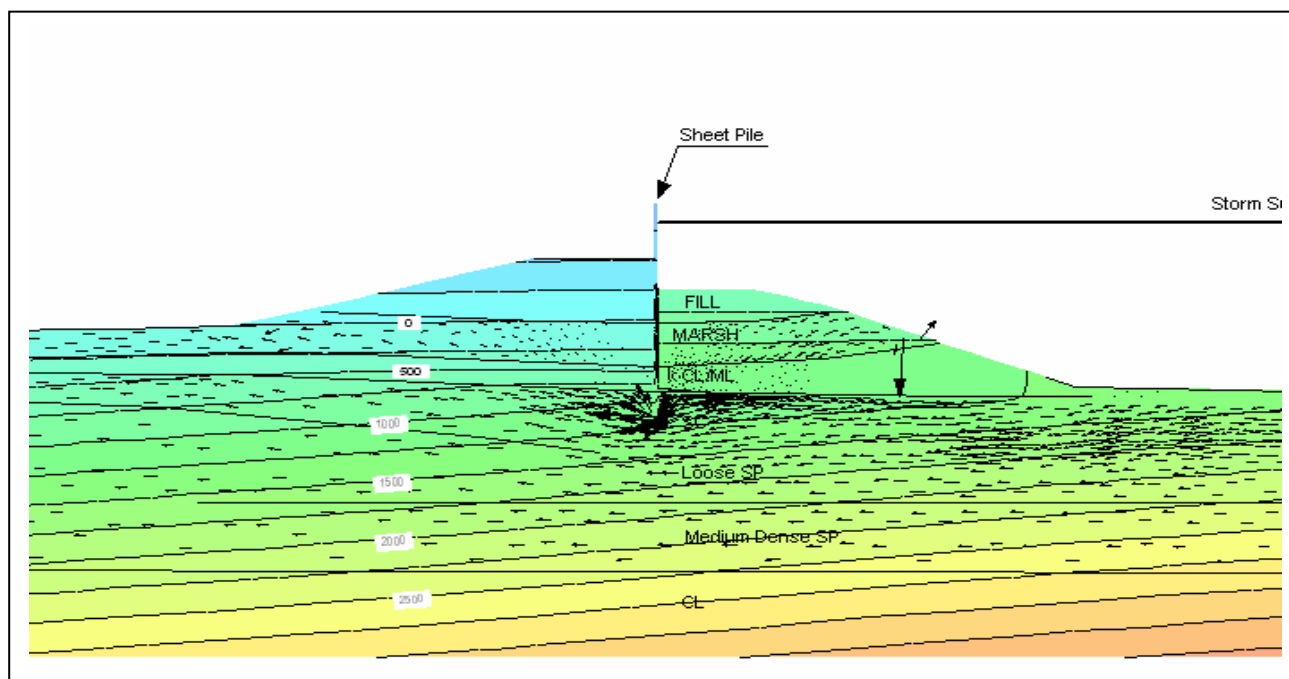


Figure 8.140: Pore water pressure contours without the gapping in the outboard toe of the floodwall, London Avenue Canal North, West bank. Storm surge at 9ft (MSL).

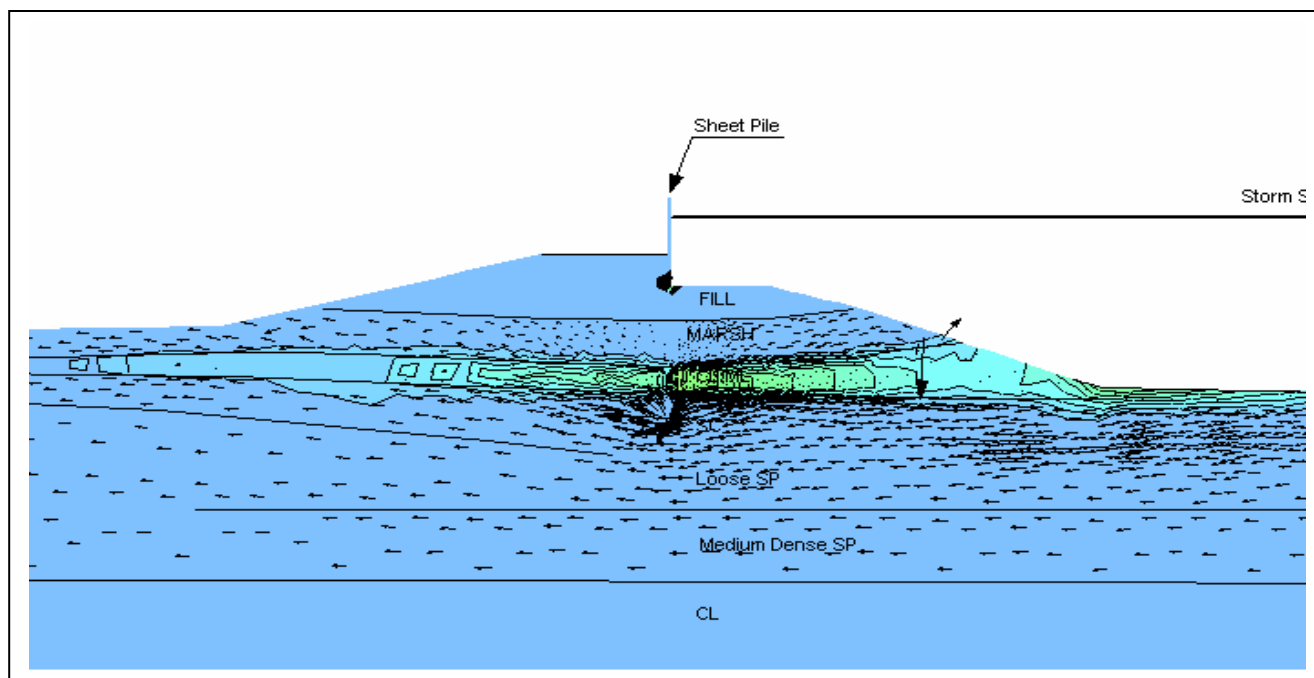


Figure 8.141: Hydraulic gradient contours without the gapping in the outboard toe of the floodwall, London Avenue Canal North, West bank. Storm surge at 9ft (MSL). Exit gradient at the inboard toe is 0.20.

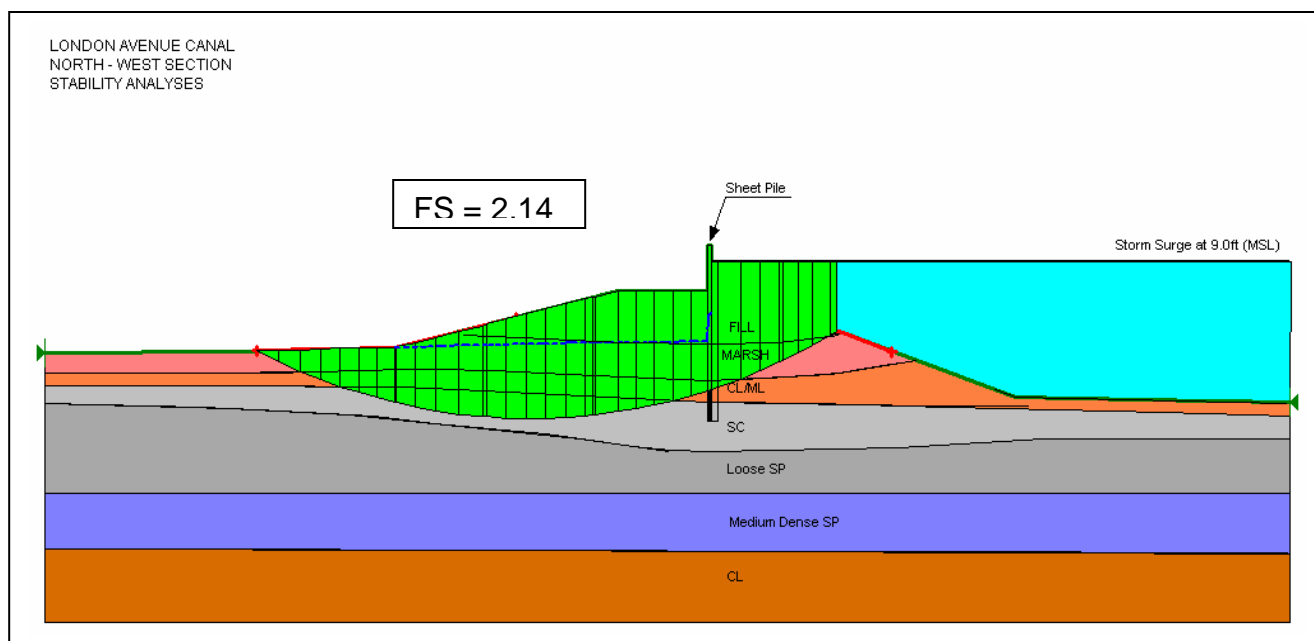


Figure 8.142: Critical failure surface without the gapping in the outboard toe of the floodwall, London Avenue Canal North, West bank. Storm surge at 9ft (MSL).

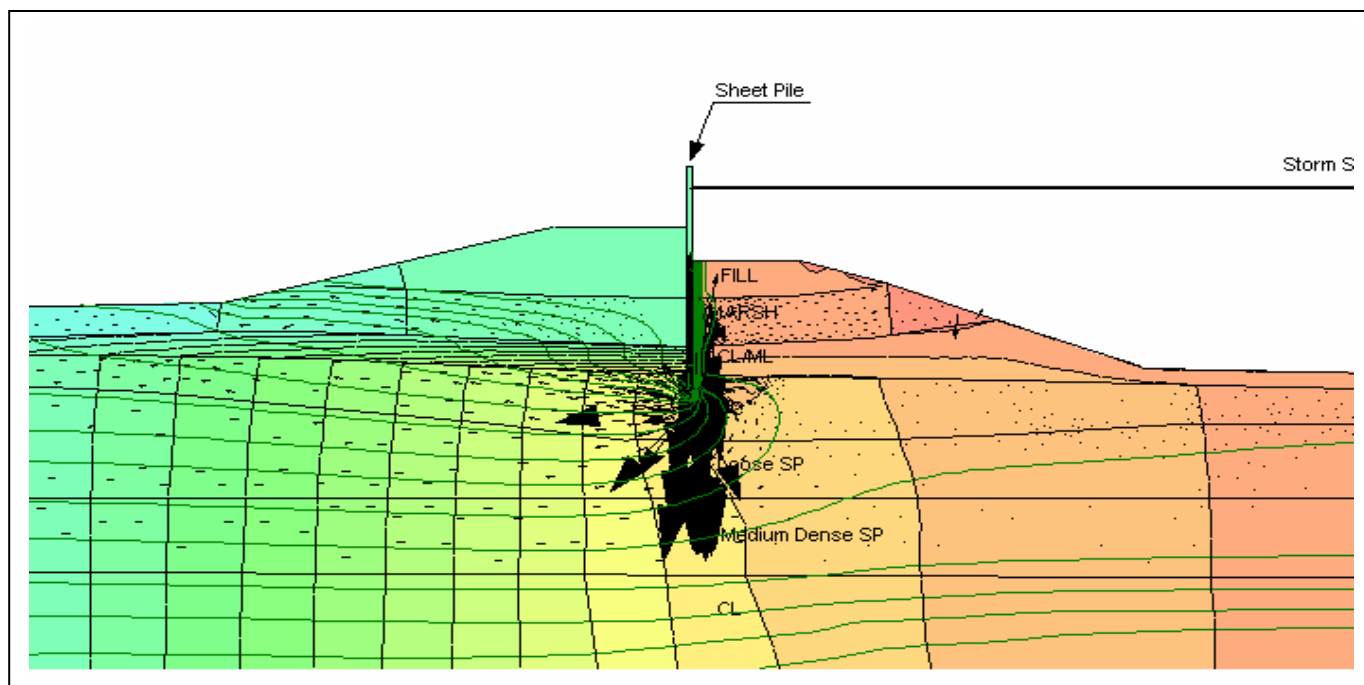


Figure 8.143: Flow net generation with the gapping in the outboard toe of the floodwall, London Avenue Canal North, West bank. Storm surge at 9ft (MSL).

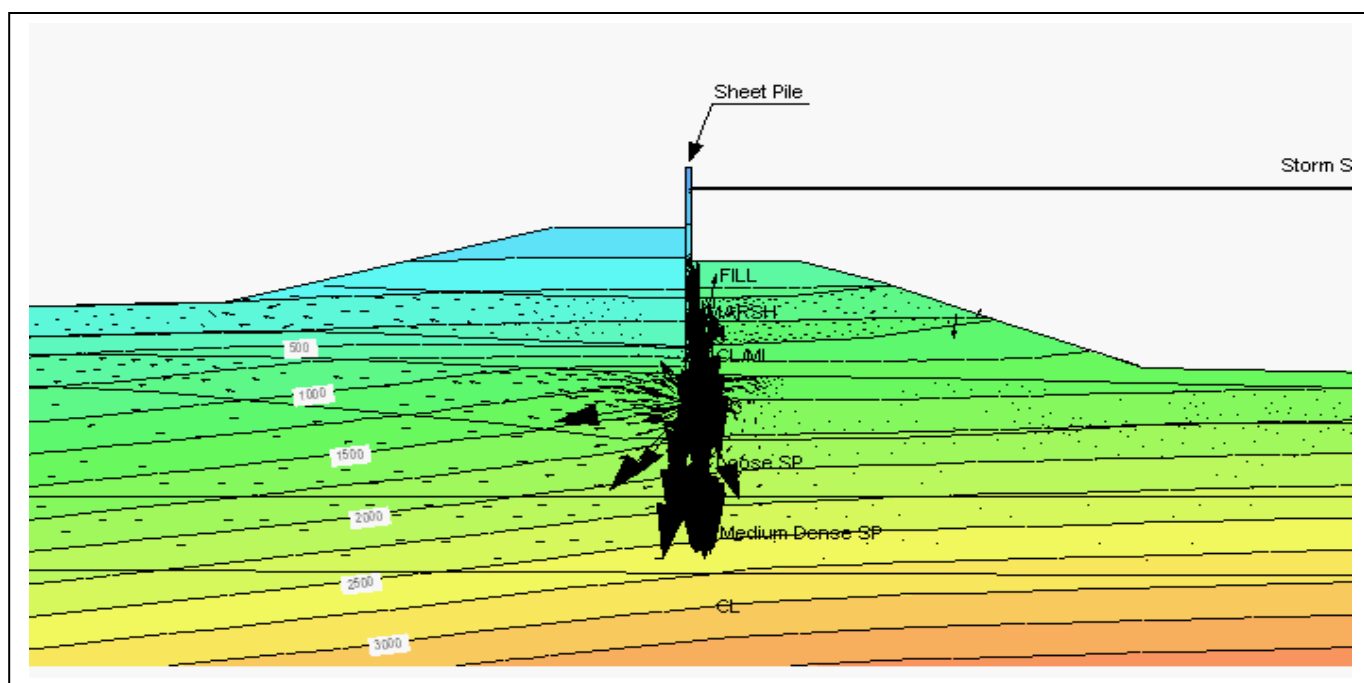


Figure 8.144: Pore water pressure contours with the gapping in the outboard toe of the floodwall, London Avenue Canal North, West bank. Storm surge at 9ft (MSL).

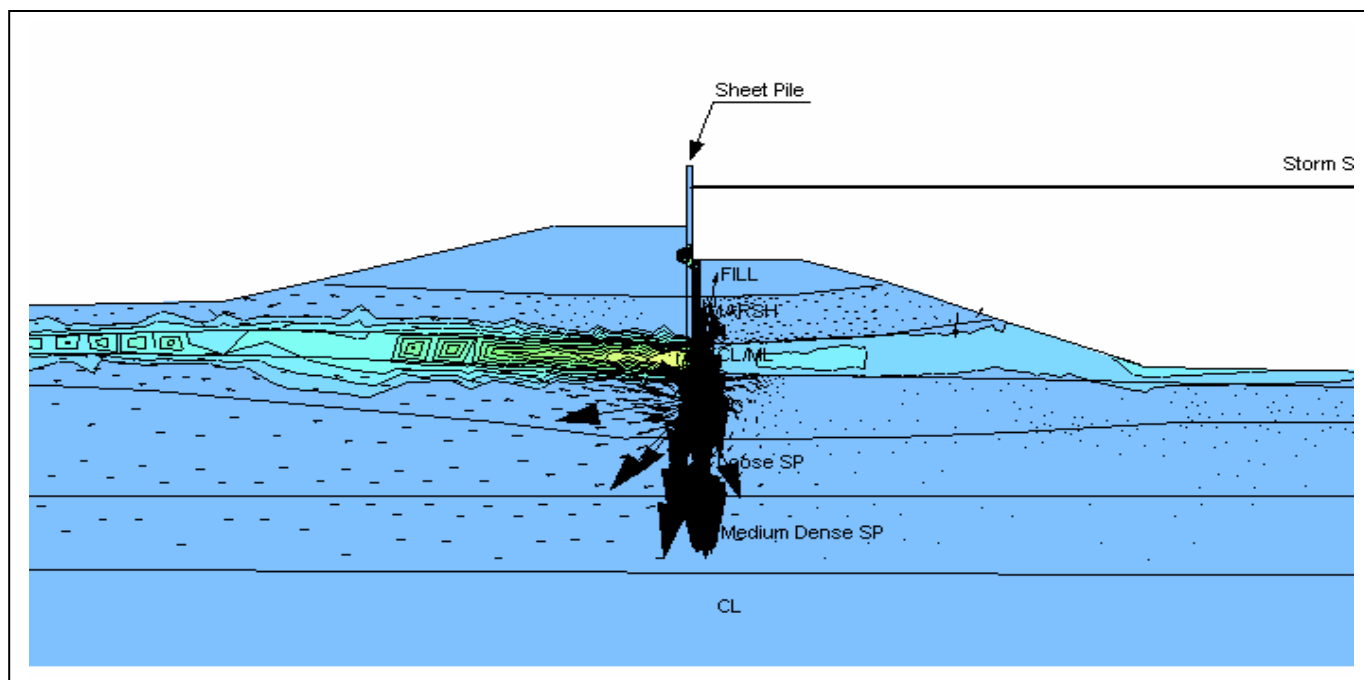


Figure 8.145: Hydraulic gradient contours with the gapping in the outboard toe of the floodwall, London Avenue Canal North, West bank. Storm surge at 9ft (MSL). Exit gradient at the inboard toe is 0.32.

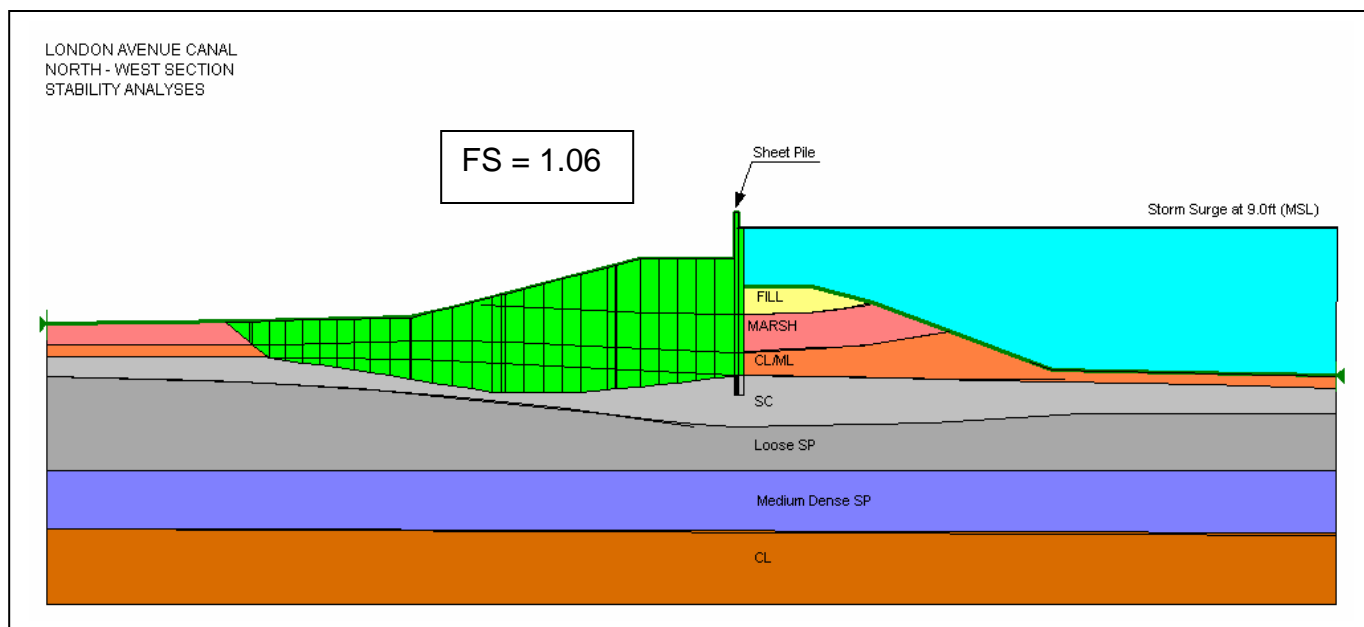


Figure 8.146: Critical failure surface with the gapping in the outboard toe of the floodwall, London Avenue Canal North, West bank. Storm surge at 9ft (MSL).

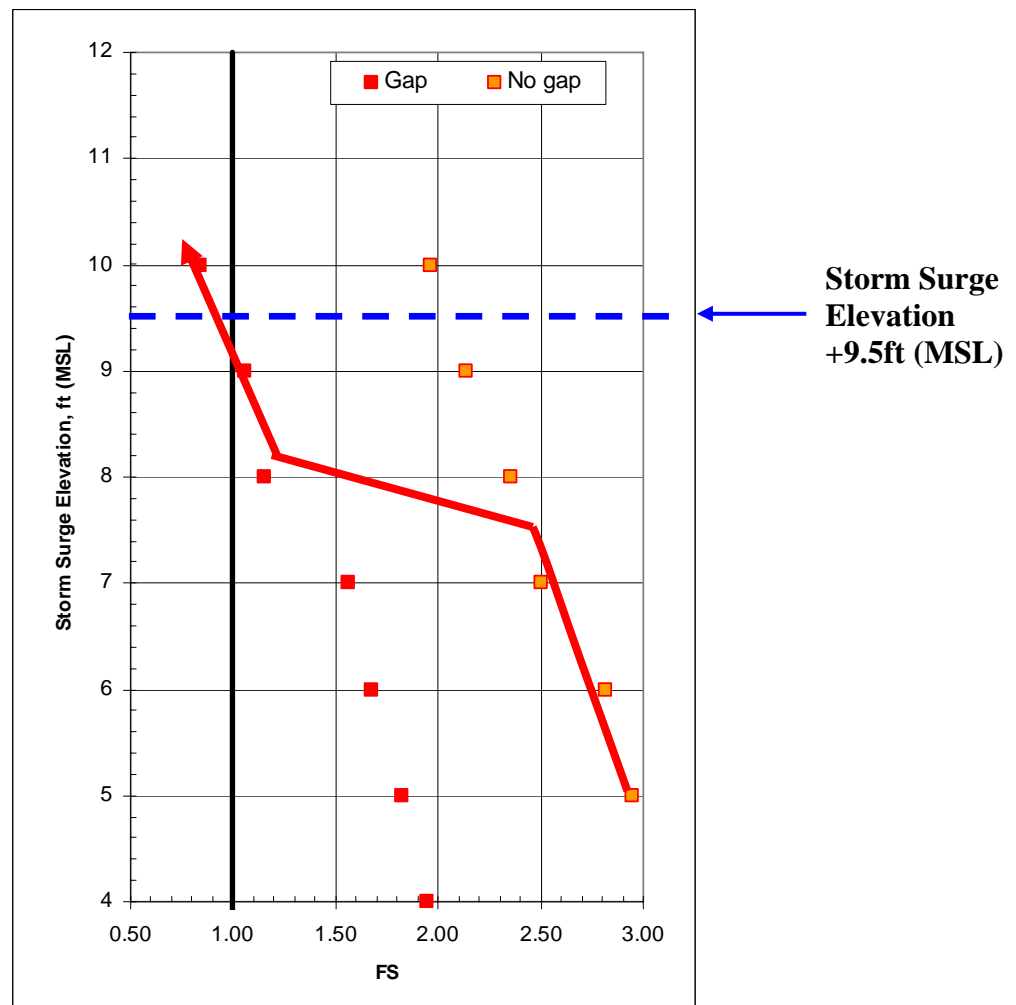


Figure 8.147 : Calculated Factors of Safety for two models based on Limit Equilibrium Analyses of the London Avenue Canal, North West bank.

ID	Soil Model	Type	γ	c	ϕ	k
			[lb/ft ³]	[lb/ft ²]	[°]	[ft/hr]
Upper fill	Mohr-Coulomb	Undrained	100	800	0	1.17E-04
Marsh, crest	Mohr-Coulomb	Undrained	80	550	0	1.17
Marsh, free field	Mohr-Coulomb	Undrained	80	300	0	1.17
CL/ML, crest	Mohr-Coulomb	Undrained	85	275	0	2.00E-04
CL/ML, free field	Mohr-Coulomb	Undrained	80	200	0	2.00E-04
SC	Mohr-Coulomb	Drained	100	0	30	0.097
Loose Sand	Mohr-Coulomb	Drained	100	0	33	0.333
Dense Sand	Mohr-Coulomb	Drained	115	0	36	0.09
Stiff Clay	Mohr-Coulomb	Undrained	95	600	0	2.00E-04

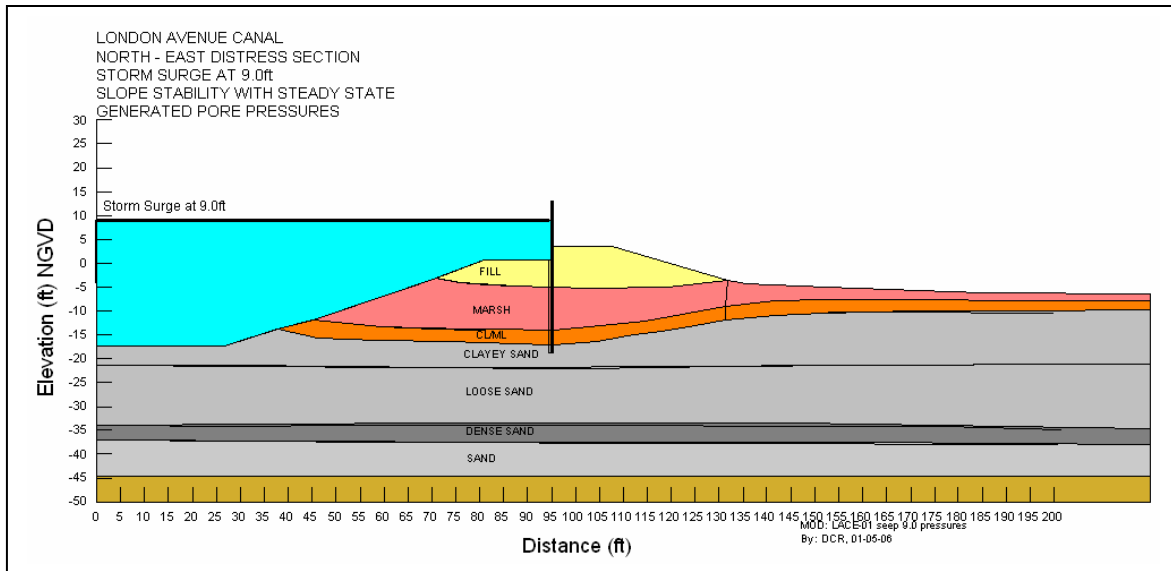


Figure 8.148: Geometry and input parameters for Limit Equilibrium and Steady State seepage Analyses for London Avenue Canal North, East bank.

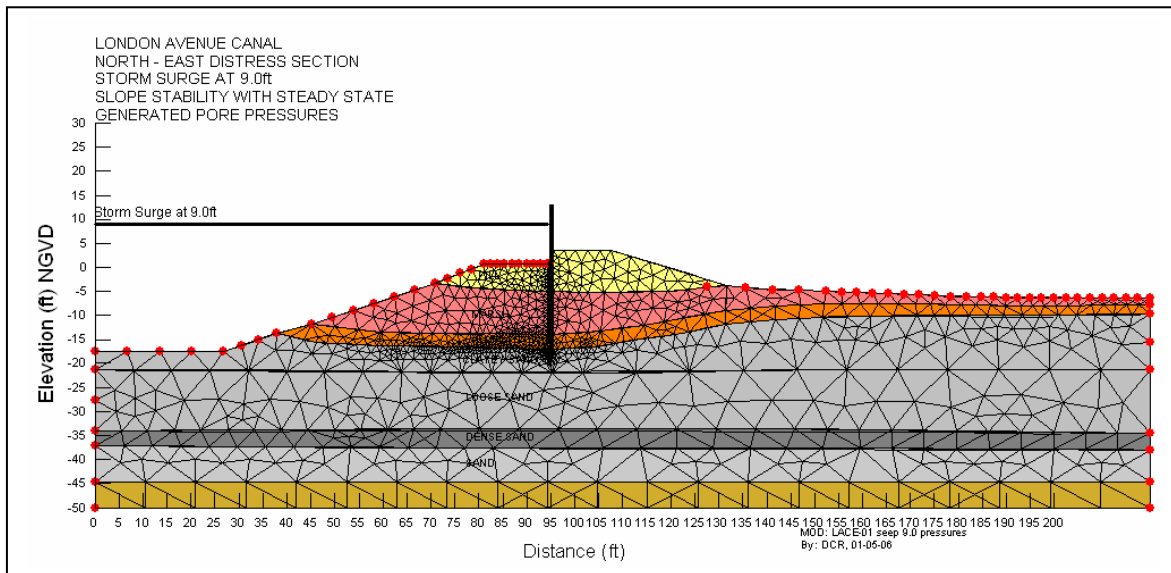


Figure 8.149: Finite Difference mesh for Steady State seepage Analyses for London Avenue Canal North, East bank.

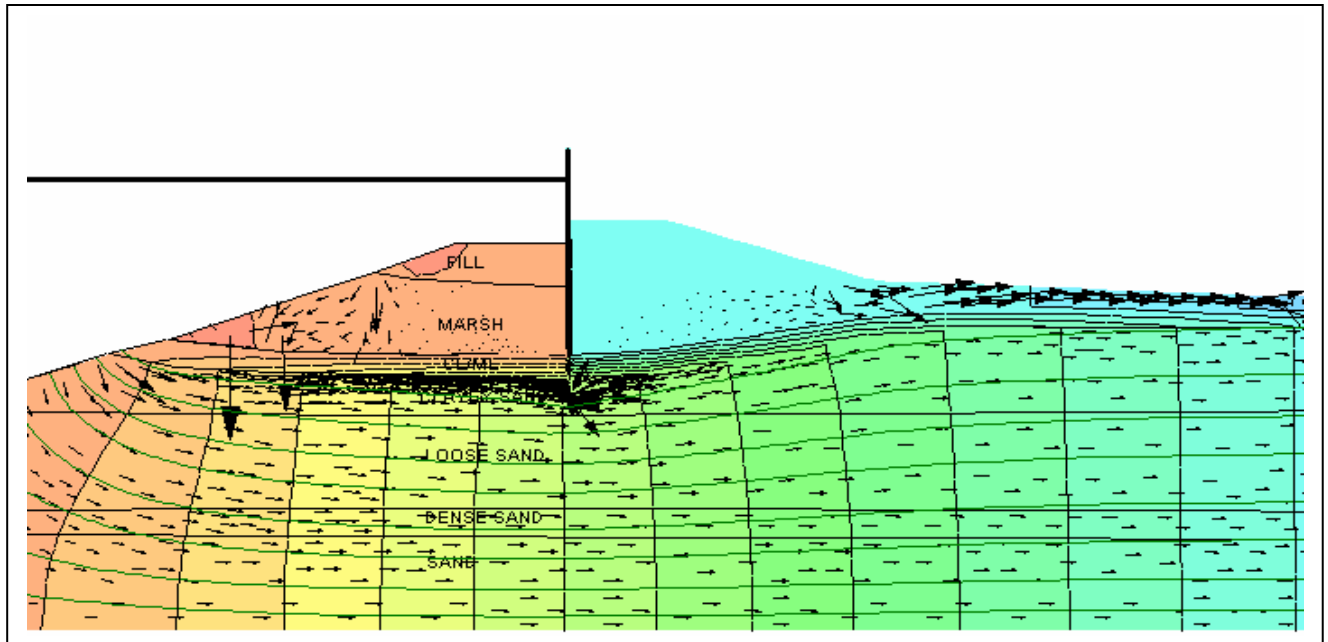


Figure 8.150: Flow net generation without the gapping in the outboard toe of the floodwall, London Avenue Canal North, East bank. Storm surge at 9ft (MSL).

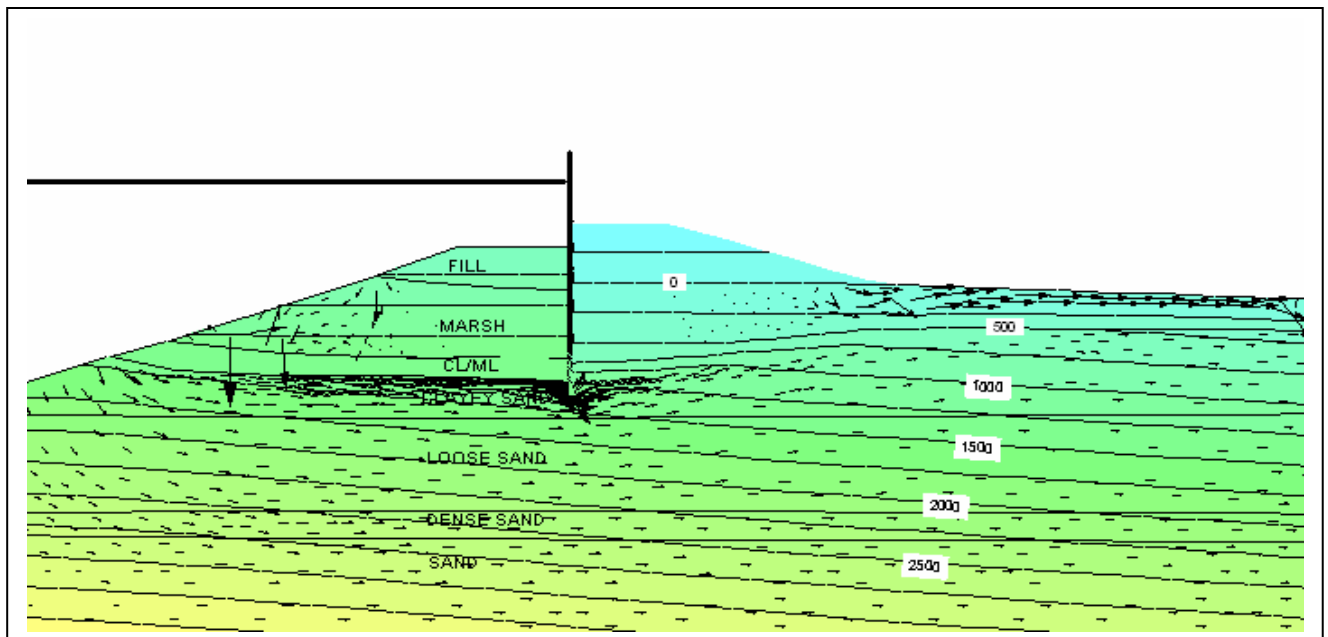


Figure 8.151: Pore water pressure contours without the gapping in the outboard toe of the floodwall, London Avenue Canal North, East bank. Storm surge at 9ft (MSL).

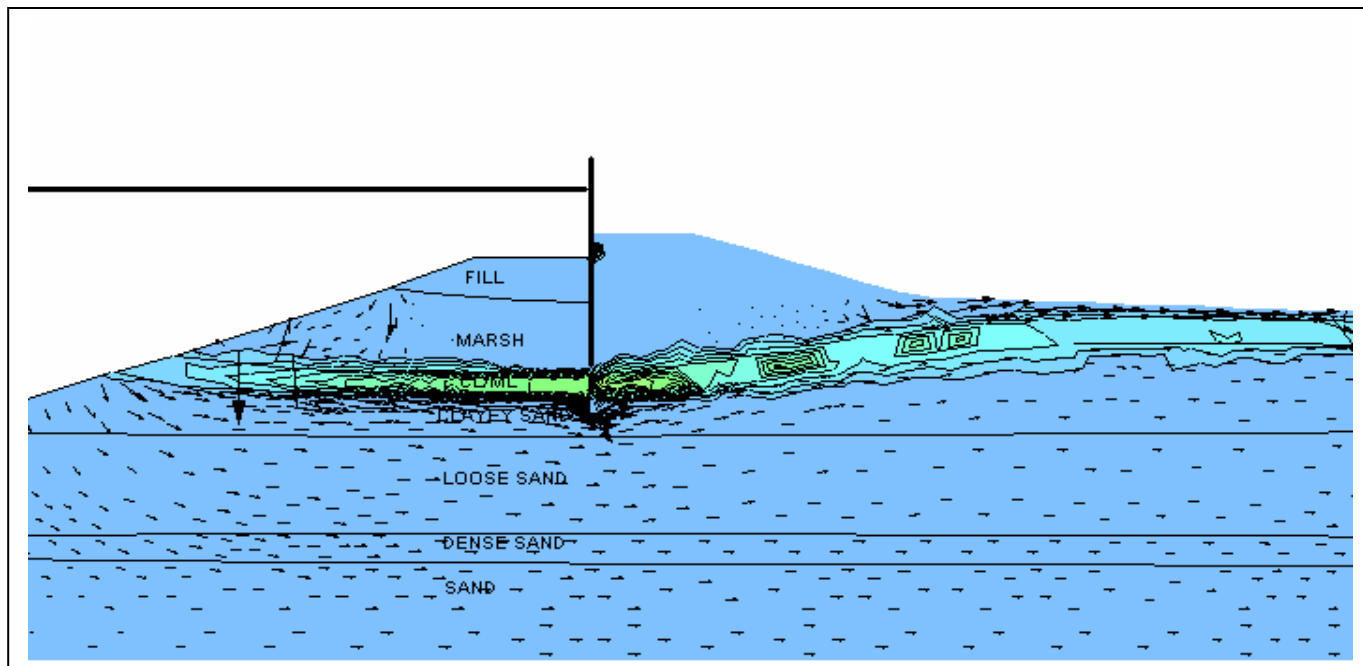


Figure 8.152: Hydraulic gradient contours without the gapping in the outboard toe of the floodwall, London Avenue Canal North, East bank. Storm surge at 9ft (MSL). Exit gradient at the inboard toe is 0.20.

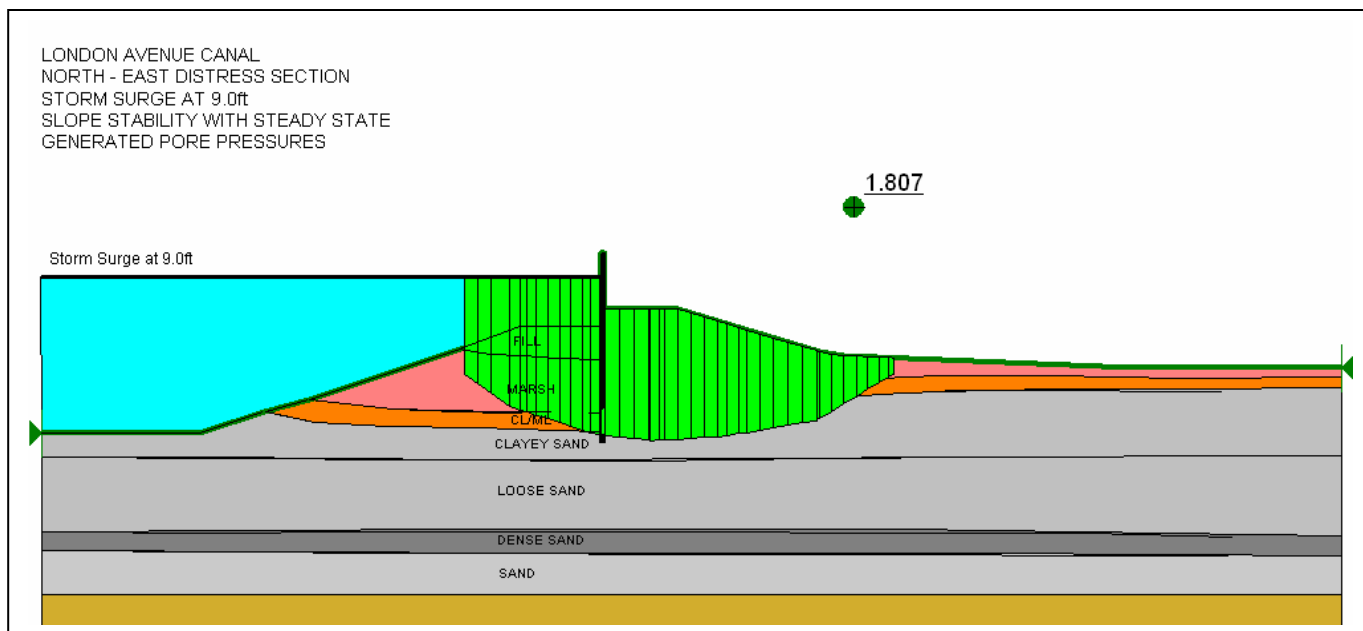


Figure 8.153: Critical failure surface without the gapping in the outboard toe of the floodwall, London Avenue Canal North, East bank. Storm surge at 9ft (MSL).

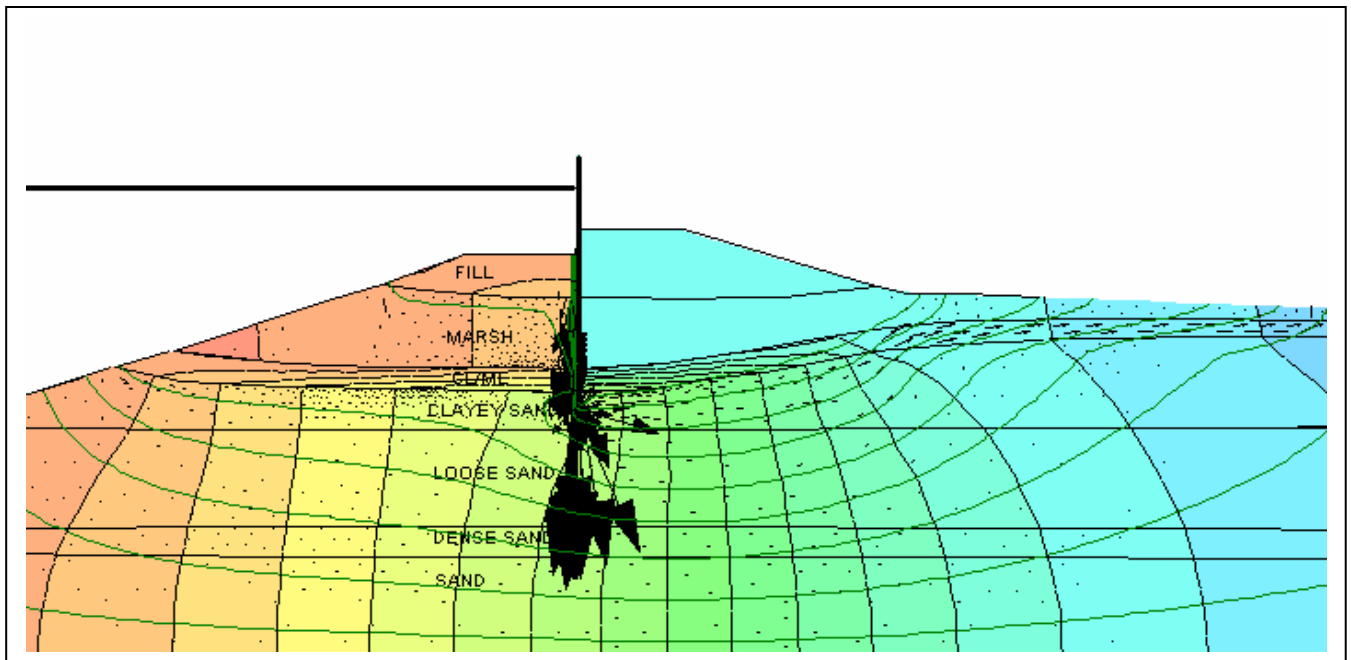


Figure 8.154: Flow net generation with the gapping in the outboard toe of the floodwall, London Avenue Canal North, East bank. Storm surge at 9ft (MSL).

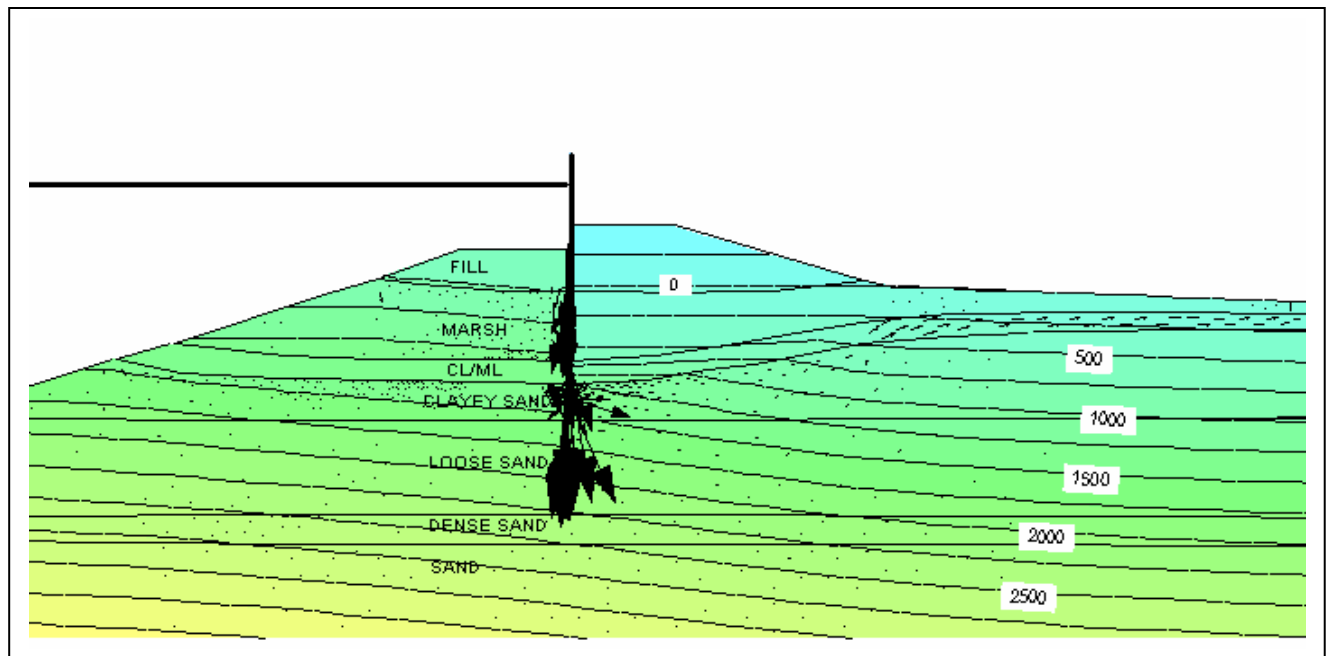


Figure 8.155: Pore water pressure contours with the gapping in the outboard toe of the floodwall, London Avenue Canal North, East bank. Storm surge at 9ft (MSL).

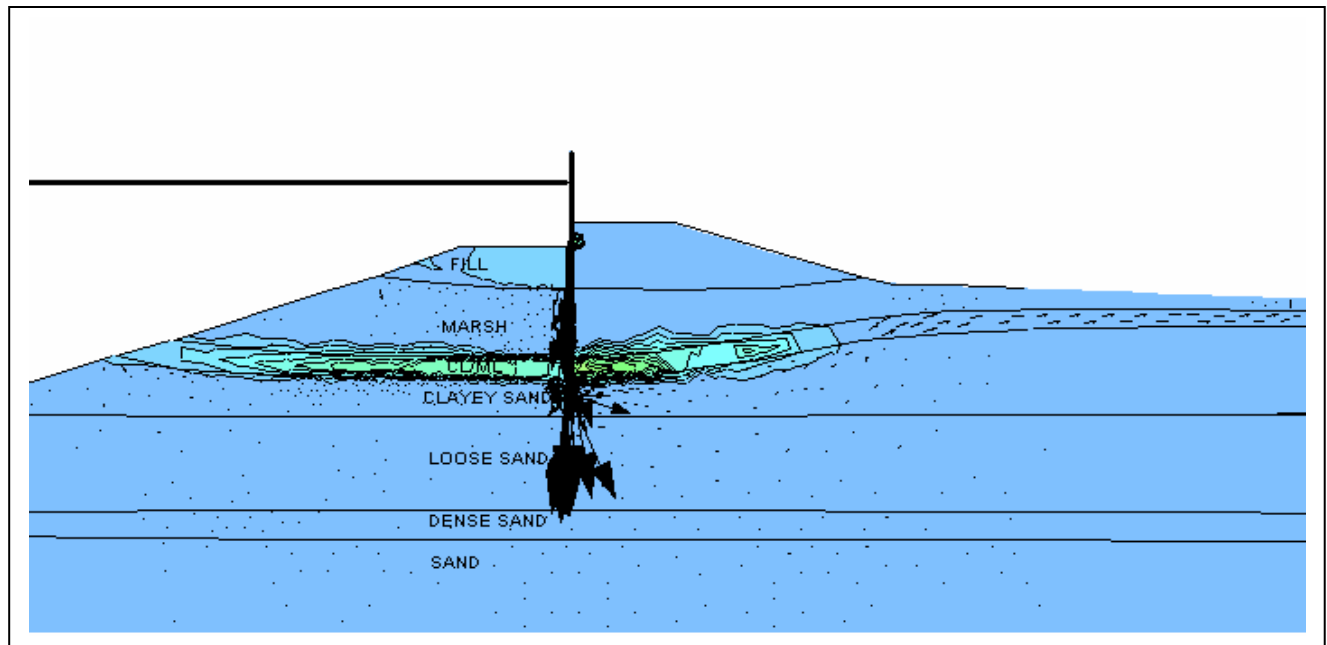


Figure 8.156: Hydraulic gradient contours with the gapping in the outboard toe of the floodwall, London Avenue Canal North, East bank. Storm surge at 9ft (MSL). Exit gradient at the inboard toe is 0.25.

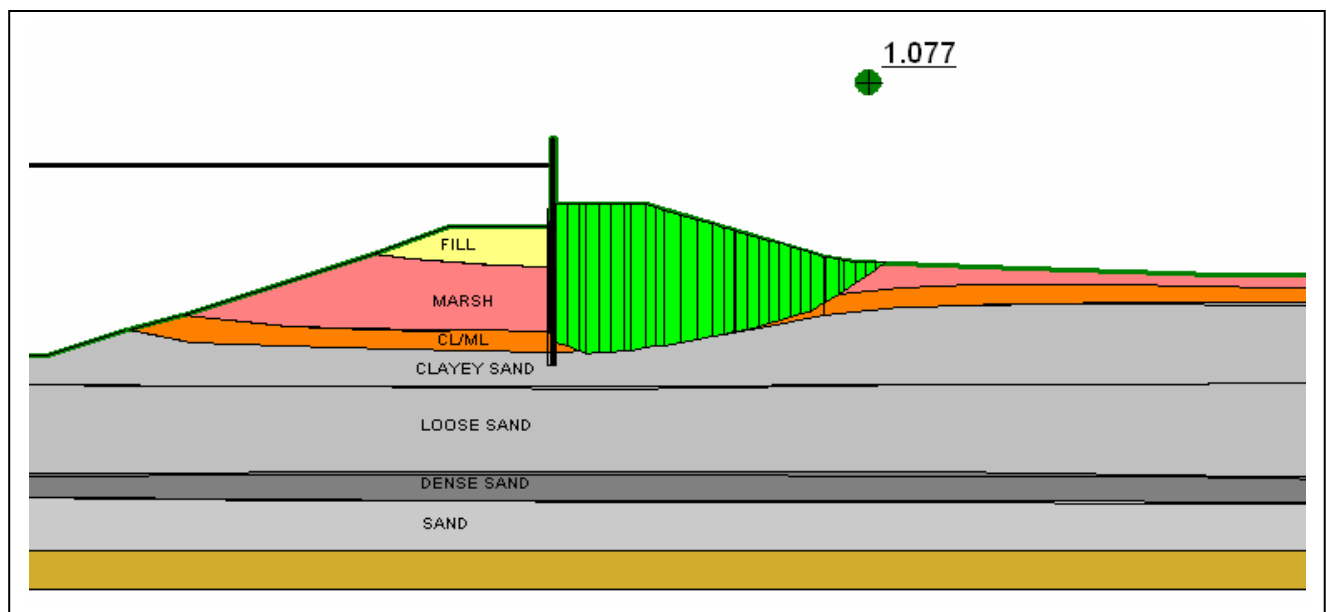


Figure 8.157: Critical failure surface with the gapping in the outboard toe of the floodwall, London Avenue Canal North, East bank. Storm surge at 9ft (MSL).

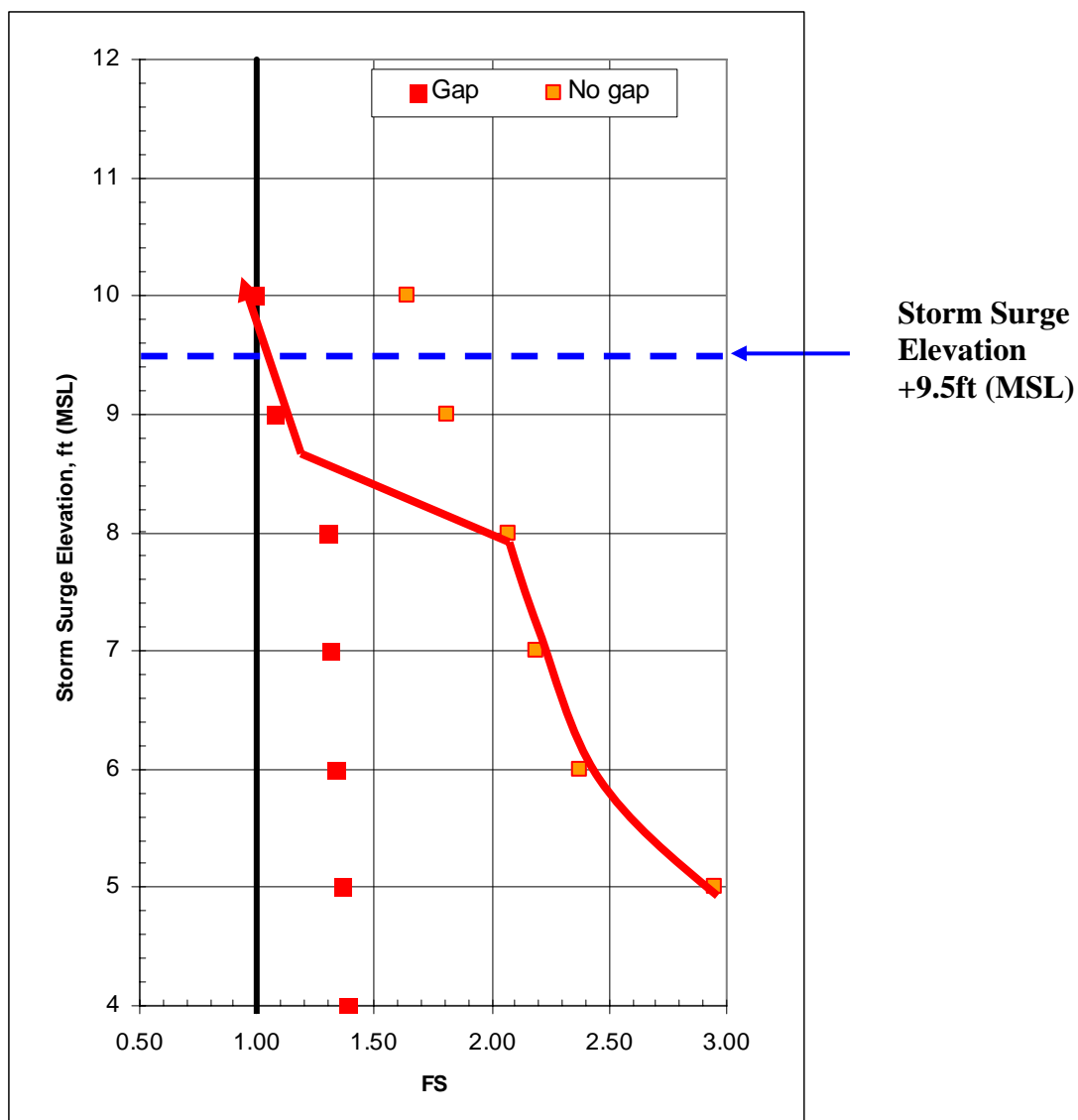


Figure 8.158 : Calculated Factors of Safety for two models based on Limit Equilibrium analyses of the London Avenue Canal, North East bank.

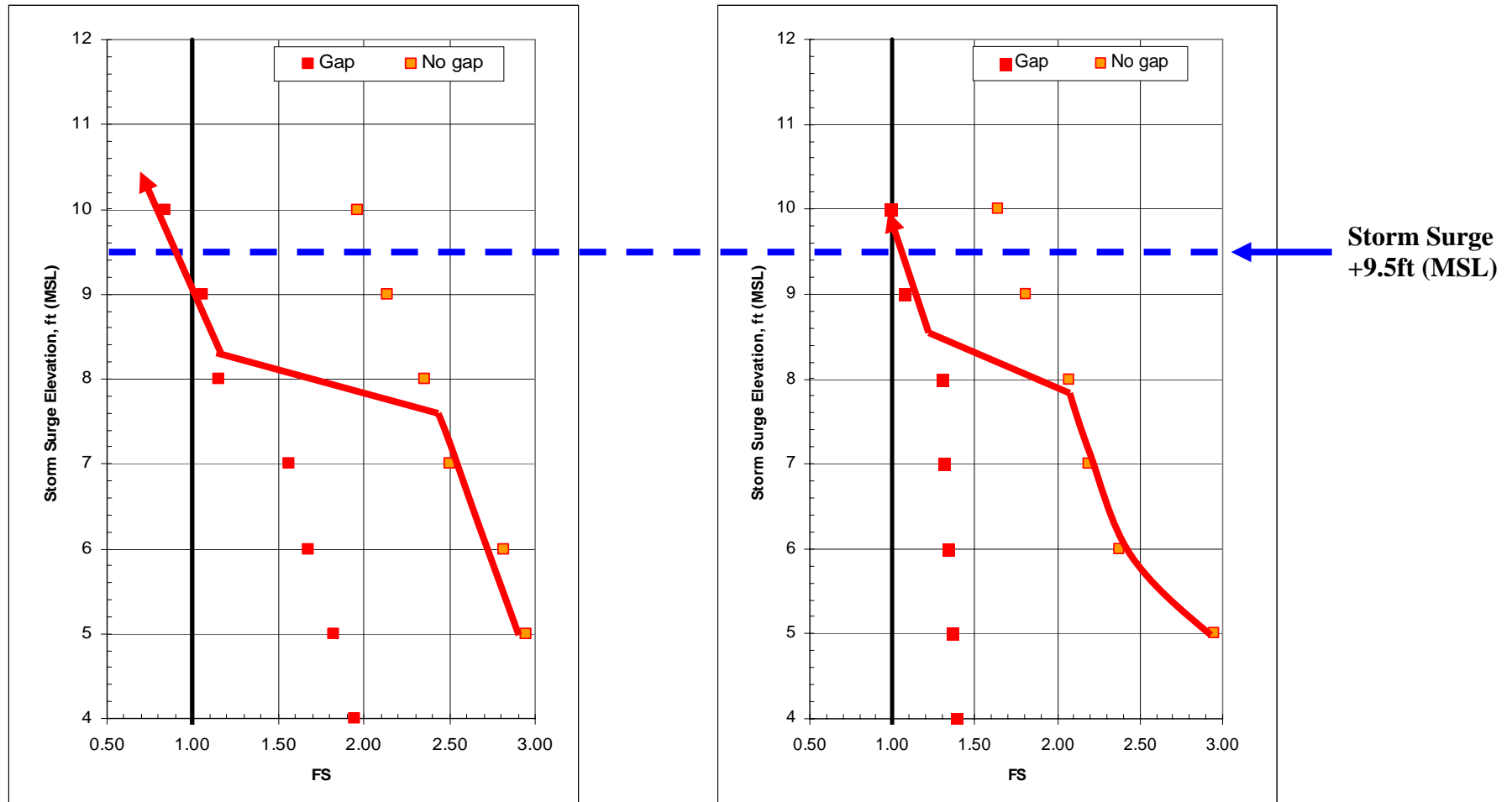


Figure 8.159: Calculated Factors of Safety for two modes based on Limit Equilibrium Analyses of the London Avenue Canal breach and distressed section for various canal water elevations; showing the best-estimated path to failure.

CHAPTER NINE: EROSION TESTS ON NEW ORLEANS LEVEE SAMPLES

9.1 Erodibility: A Definition

Erodibility is a term often used in scour and erosion studies. Erodibility may be thought of as one number which characterizes the rate at which a soil is eroded by the flowing water. With this concept erosion resistant soils would have a low erodibility index and erosion sensitive soils would have a high erodibility index. This concept is not appropriate; indeed the water velocity can vary drastically from say 0 m/s to 5 m/s or more and therefore the erodibility is not a single number but a relationship between the velocity applied and the corresponding erosion rate experienced by the soils. While this is an improved definition of erodibility, it still presents some problems because water velocity is a vector quantity which varies everywhere in the flow and is theoretically zero at the soil water interface. It is much preferable to quantify the action of the water on the soil by using the shear stress applied by the water on the soil at the water-soil interface. Erodibility is therefore defined here as the relationship between the erosion rate \dot{z} and the hydraulic shear stress applied τ (Figure 9.1). This relationship is called the erosion function $\dot{z}(\tau)$. The erodibility of a soil or a rock is represented by the erosion function of that soil or rock. This erosion function can be obtained by using a laboratory device called the EFA (Erosion Function Apparatus) and described later.

9.2 Erosion Process

Soils are eroded particle by particle in the case of coarse-grained soils (cohesionless soils). In the case of fine-grained soils (cohesive soils), erosion can take place particle by particle but also block of particles by block of particles. The boundaries of these blocks are formed naturally in the soil matrix by micro-fissures due to various phenomena including compression and extension.

The resistance to erosion is influenced by the weight of the particles for coarse grained soils and by a combination of weight and electromagnetic and electrostatic inter-particle forces for fine grained soils. Observations at the soil water interface on slow motion videotapes indicates that the removal of particle or blocks of particles is by a combination of rolling and plucking action of the water on the soil.

9.3 Velocity vs. Shear Stress

The scour process is highly dependent on the shear stress developed by the flowing water at the soil-water interface. Indeed, at that interface the flow is tangential to the soil surface regardless of the flow condition above it; very little water if any flows perpendicular to the interface. The water velocity in the river is in the range of 0.1 to 3 m/s, whereas the bed shear stress is in the range of 1 to 50 N/m² (Figure 9.2) and increases with the square of the water velocity. The magnitude of this shear stress is a very small fraction of the undrained shear strength of clays used in foundation engineering (Figure 9.3).

It is interesting to note that such small shear stresses are able to scour rocks to a depth of 1,600 m, as in the case for the Grand Canyon over the last 20 million years at an average scour rate of 9.1×10^{-6} mm/hr. This leads one to think that even small shear stresses if applied cyclically by the turbulent nature of the flow can overcome, after a sufficient number of cycles, the crystalline bonds in a rock and the electromagnetic bonds in a clay. This also leads one to think that there is no cyclic stress threshold, but that any stress is associated with a number of cycles to failure. (Gravity bonds seem to be an exception to this postulate, because it appears that gravity bonds cannot be weakened by cyclic loading.) This postulate contradicts the critical shear stress concept discussed later.

The profile of the water velocity versus depth in the flow (Figure 9.4) indicates a maximum velocity at the free surface and a zero velocity at the bottom of the flow. This zero velocity boundary is due to the fact that the water does not flow below the flow bottom. While the velocity is zero at the bottom, the shear stress is maximum because the shear stress is proportional to the slope of the velocity profile versus depth. This is explained in Figure 9.4. One can think of the water element in contact with the bottom as a simple shear test on water. Since water is a Newtonian fluid, the shear stress that it develops is proportional to the rate at which it is sheared. This governs the equation in Figure 9.4.

9.4 Erosion Threshold and Erosion Categories

The critical velocity is the velocity at which the soil starts to erode. The critical shear stress is the shear stress at which the soil starts to erode (Figures 9.1 and 9.2). Below these values there is no erosion, above these values the soil erodes at a certain rate. This threshold of erosion is very useful in engineering but it is not obvious that such a clear threshold truly exists physically. Indeed a sample of granite, for example, has a very high critical shear stress. Yet common sense tells us that a pebble made of granite and left under a dripping faucet for 20 million years would develop a hole. In this case, the critical shear as we conceive it would not have been reached yet the rock would have been eroded. The reason for the hole in the pebble may be that there is no such thing as a cyclic threshold for materials and that cyclic stresses even very small can destroy any material bonds; it is only a matter of the number of cycles to break the bond. So one has to accept a practical definition of the critical shear stress. The critical shear stress is defined here as the shear stress corresponding to a rate of erosion of 1 mm/hr in the Erosion Function Apparatus. Values of critical shear stresses are shown in Figure 9.2.

If the critical shear stress is exceeded, it becomes important to know how fast the soil is eroding at a given velocity. The relationship between the erosion rate and the velocity or the interface shear stress is a function. In order to quantify this erosion function using a single number, the following scheme is proposed. It consists in placing the erosion function on the erosion chart of Figure 9.5 and deciding what erodibility category fits best for the soil considered. This approach holds promise to use only one number to characterize a function. Work is ongoing to tie a number of soil types into erodibility categories.

9.5 Erodibility of Coarse-Grained Soils

Clean sands and gravels erode particle by particle. This has been observed on slow motion videotapes. Three mechanisms seem to be possible: sliding, rolling, and plucking.

A simple sliding mechanism (Figure 9.6) consists of assuming that the soil particle is a sphere, that the resultant force exerted by the water on the soil particle is a shear force parallel to the eroding surface, and that neighboring particles do not exert forces on the particle being analyzed because they move at the same rate. Electromagnetic and electrostatic forces between particles are neglected because the analysis is done for a sand or a gravel particle. As the velocity increases, the shear stress τ imposed by the water on the particle becomes large enough to overcome the friction between two particles stacked on top of each other, and sliding takes place. The critical shear stress τ_c is the threshold shear stress at which erosion is initiated. Referring to Figure 9.6, horizontal equilibrium leads to (White 1940):

$$\tau_c A_e = W \tan \phi \quad (1)$$

where A_e = effective friction area of the water on the particle; W = submerged weight of the particle; and ϕ = friction angle of the interface between two particles. If the particle is considered to be a sphere, (1) can be rewritten as

$$\tau_c \alpha (\pi D_{50}^2/4) = (\rho_s - \rho_w) g (\pi D_{50}^3/6) \tan \phi \quad (2)$$

$$\text{or} \quad \tau_c = 2 (\rho_s - \rho_w) g D_{50} (\tan \phi)/3\alpha \quad (3)$$

where α = ratio of the effective friction area over the maximum cross section of the spherical particle; D_{50} = mean diameter representative of the soil particle size distribution; ρ_s and ρ_w = mass density of the particles and of water, respectively; and g = acceleration due to gravity. Eq. (3) shows that the critical shear stress is linearly related to the particle diameter. Briaud et al. (1999b) showed experimentally for a sand and a gravel tested in the EFA that an approximate relationship is:

$$\tau_c \text{ (N/m}^2\text{)} = D_{50} \text{ (mm)} \quad (4)$$

Using (3) and (4), and assuming reasonable values for ρ_s , ρ_w , g , and ϕ , leads to a value of α equal to about 6. This value is many times higher than would be expected and shows that the sliding mechanism is not the eroding mechanism, or at least not the only one involved.

A simple rolling mechanism (Figure 9.7) consists of assuming that the soil particle is a sphere, that the resultant force exerted by the water on the soil particle is a shear force parallel to the eroding surface, that neighboring particles do not impede the process, and that rotation takes place around the contact point with the underlying particle. Electromagnetic and electrostatic forces between particles are neglected because the analysis is done for a sand or a gravel particle. At incipient motion and referring to Figure 9.7, moment equilibrium around the contact point O leads (White 1940) to:

$$\tau_c A_e a = W b \quad (5)$$

$$\text{or} \quad \tau_c (\alpha \pi D_{50}^2/4) (D_{50}/2 + D_{50}(\cos\beta)/2) = (\rho_s - \rho_w) g (\pi D_{50}^3/6) (D_{50}(\sin\beta)/2) \quad (6)$$

$$\text{or} \quad \tau_c = 2 ((\rho_s - \rho_w) g D_{50} \sin\beta)/(3\alpha (1 + \cos\beta)) \quad (7)$$

Eq. (7) confirms that τ_c is linearly proportional to D_{50} . For reasonable values of ρ_s , ρ_w , and g , and for $\alpha = 1$, using (4) and (7) leads to β values equal to about $10-12^\circ$, which is indicative of a loose arrangement; indeed, the sand and the gravel tested to obtain Eq. 4 were placed in a very loose condition in the EFA. Therefore it appears that rolling is more reasonable a mechanism than sliding. Equation 7 tends to indicate that, while τ_c is linearly proportional to D_{50} , the proportionality factor may depend on the relative density. The dominant value of the

angle β can be obtained from a contact angle distribution diagram such as the ones shown in Figure 9.8.

A simple plucking mechanism consists of assuming that the particles are cubes with a side a . The water pressure on top of the cube is u_t and the water pressure at the bottom of the cube is u_b . If it is assumed that all particles are plucked up at the same time, the differential pressure between the top and bottom necessary to initiate plucking of the particle or block of particles is:

$$W = (u_b - u_t) a^2 \quad (8)$$

$$\text{or} \quad \rho_s g a = u_b - u_t \quad (9)$$

The differential pressure $u_b - u_t$ is made up of the hydrostatic differential water pressure $(u_b - u_t)_o$ and the differential water pressure created by the flow Δu

$$\text{or} \quad u_b - u_t = (u_b - u_t)_o + \Delta u \quad (10)$$

For a particle with $a = 1$ mm, the hydrostatic differential water pressure $(u_b - u_t)_o$ is 10 N/m^2 . This hydrostatic differential water pressure reduces the weight of the particle to its buoyant weight. The additional differential water pressure necessary to pluck the particle away Δu is 15 N/m^2 . This value of Δu is equivalent to 1.5 mm of water and it is easy to conceive that such a small differential pressure can be developed. It is created dynamically by the water flow including the fluctuations and the turbulence in the water. These pressure fluctuations are very difficult to measure (Einstein and El-Samni, 1949 and Apperley, 1968). These pressure fluctuations can be calculated through advanced numerical simulations.

These simplistic analyses of the sliding, rolling, and plucking mechanisms help to clarify the important factors affecting the incipient motion of coarse grained soils. However, they are not reliable for prediction purposes, and today experiments are favored over theoretical expressions to determine τ_c for example. Shields (1936) ran a series of flume experiments with water flowing over flat beds of sands. He plotted the results of his experiments in a dimensionless form on what is now known as the Shields diagram. This data as well as other data on sand gathered at Texas A&M University are plotted in Figure 9.2 as critical shear stress τ_c versus mean grain size D_{50} . Eq. (4) is shown in Figure 9.2 and seems to fit well for sands. Shields did not perform any experiments on silts and clays. The data developed for silts and clays at Texas A&M University show that Eq.(4) is not applicable to fine grained soils and that D_{50} is not a good predictor of τ_c for those types of soils.

There seems to be consensus in using the shear stress applied by the water to the soil at the soil water interface as the major parameter causing erosion. It is likely that the hydraulic normal stress or pressure created by the water at that interface also contributes to the process.. Nevertheless, the use of the shear stress only has remained common practice and the role of the normal stress that generates bursts of uplift forces during turbulent flow has yet to be included in common approaches to scour.

9.6 Erodibility of Fine-Grained Soils

In the case of silts and clays, other forces come into play besides the weight of the particles; these are the electrostatic and Van der Waals forces. Figure 9.10 and 9.11 show cartoons of the forces and pressures acting on the soil particle in the general case. The water

pressure u_w surrounds the particle if the soil is saturated. The contact forces f_{ci} exist at the contact point and have normal as well as shear components. The electrostatic and Van der Waals forces f_{ei} are also shown on the figure. Figure 9.10 refers to the case where the water is not moving. In this case the water pressure is smaller on the top of the particle than on the bottom of the particle but the difference is not significant. This difference is equal to the hydrostatic pressure difference due to the height of the particle and creates the buoyancy of the soil particle. In Figure 9.11, the water is moving and the difference between the top and bottom water pressure has increased. Note that the water pressure u_w and therefore the uplift force on the particle is a function of time t and fluctuates during the flow. The cartoon shows a situation where the water pressure may be such that the particle weight is overcome.

The electrostatic forces are likely to be repulsive because clay particles are negatively charged. Van der Waals forces are relatively weak electromagnetic forces that attract molecules to each other (Mitchell 1993); although electrically neutral, the molecules form dipoles that attract each other like magnets. The Van der Waals forces are the forces that keep H_2O molecules together in water. The magnitude of these Van der Waals forces can be estimated by (after Black et al. 1960):

$$f(N/m^2) = 10^{-28} / d(m)^4 \quad (11)$$

where $d(m)$ = distance in m between soil particles; and f = attraction force in N/m^2 . By multiplying f by the particle surface area, one can obtain the inter-particle force. Table 9.1 shows the value of these forces for a sand and a clay particle.

In both cases the soil particle was assumed to be spherical and the distance between particles was taken equal to the particle diameter. While such an evaluation of the Van der Waals force can only be considered as a crude estimate, the following observations regarding the numbers in Table 9.1 are interesting. First, the ratio between the weight and size of the sand particle and the clay particle are similar to the ratio between the weight and size of a Boeing 747 and a postage stamp; therefore, if the critical shear stress is proportional to the particle weight, the critical shear stress for clays should be practically zero. Second, the ratio between the Van der Waals force and the weight of the sand particle indicates that the Van der Waals force is truly negligible for sands. Third, the same ratio for the clay particle, while 10^{17} times larger than for sand, also indicates that the Van der Waals forces are negligible compared with the weight of the clay particle. This would lead one to think that the critical shear stress, τ_c , is essentially zero for clays. Note that the electrostatic forces have not been calculated here but since they are predominantly repulsive they would decrease, if anything, the attraction due to the Van der Waals forces. Other phenomena give cohesion to clays; they include water meniscus forces, such as those developing when a clay dries, and diagenetic bonds due to aging, such as those developing when a clay turns into rock under pressure over geologic time. Because of the number and complexity of these bonds, it is very difficult to predict τ_c for clays empirically on the basis of a few index properties. Several researchers however have proposed empirical equations for τ_c in clays, such as Dunn (1959) and Lyle and Smerdon (1965).

One problem associated with measuring τ_c is determining the initiation of scour. When the particles are visible to the naked eye, it is simple to detect when the first particle is scoured away. For clays this is not the case, and various investigators define the initiation of

scour through different means; these vary from “when the water becomes muddy” to extrapolation of the scour rate versus shear stress curve back to zero scour rate. Table 9.2 shows a variety of measured τ_c values. The lack of precise definition for the initiation of scour may be in part responsible for the wide range of values.

Beyond the critical shear stress, a certain scour rate z' (mm/hr) is established. This scour rate is rapid in sand, slow in clay, and extremely slow in rock. The example of the Grand Canyon rock cited earlier leads to a value of z' equal to 9.1×10^{-6} mm/hr, whereas fine sands erode at rates of 10^4 mm/hr as measured in the EFA. Clays scour at intermediate rates with common values in the range of 1 to 1,000 mm/hr. The high scour rate in sand exists because once gravity is overcome, no other force slows the scour process down. The very low scour rate in rock exists probably because it takes a large number of shear-stress cycles imposed by the turbulent nature of the flow to overcome the very strong crystalline bonds binding the rock together. Note that rock scour can also occur at larger rates if the rock is fractured and the water flow provides very high velocities as in the case of the downstream end of high dam spillways. The low scour rate in clays is probably associated with the fact that it takes a large number of shear stress cycles to overcome the electromagnetic bonds created by the Van der Waals forces between clay particles. Even though these bonds are relatively weak, as discussed previously, they are sufficient to slow the scour process significantly. The scour rate z' versus shear stress τ_c curve (Figure 9.1) is used to quantify the scour rate of a soil as a function of the flow. Several researchers have measured the rate of erosion in cohesive soils; most have proposed a straight line variation (Ariathurai and Arulanandan, 1978), while some have found S shape curves (Christensen, 1965). Some of the rates quoted in the literature are given in Table 9.3.

Some of the factors influencing the erodibility of fine grained soils are listed in Table 9.4. Although there are sometimes conflicting findings, the influence of various factors on cohesive soil erodibility is shown in Table 9.4 when possible.

The critical shear stress of coarse grained soils is tied to the size of the particles and usually ranges from 0.1 N/m^2 to 5 N/m^2 . The rate of erosion of coarse grained soils above the critical shear stress increases rapidly and can reach tens of thousands of millimeters per hour. The most erodible soils are fine sands and silts with mean grain sizes in the 0.1 mm range (Figure 9.2). The critical shear stress of fine grained soils is not tied to the particle size but rather to a number of factors as listed in Table 9.4. The critical shear stress of fine grained soils however varies within the same range as coarse grained soils (0.1 N/m^2 to 5 N/m^2) for the most common cases. One major difference between coarse grained and fine grained soils is the rate of erosion beyond the critical shear stress. In fine grained soils (often called cohesive soils), this rate increases slowly and is measured in millimeters per hour. This slow rate makes it advantageous to consider that erosion problems are time dependent and to find ways to accumulate the effect of the complete velocity history rather than to consider a design flood alone.

9.7 Erodibility and Correlation to Soil Properties

There is a critical shear stress τ_c below which no erosion occurs and above which erosion starts. This concept while practically convenient may not be theoretically simple.

Indeed, as seen on Figure 9.1, there is no obvious value for the critical shear stress. The critical shear stress is arbitrarily defined as the shear stress which corresponds to an erosion rate of 1 mm/hr. The critical shear stress is associated with the critical velocity v_c . One can also define the initial slope $S_i = (dz/dt)_i$ at the origin of the erosion function. Both τ_c and S_i are parameters which help describe the erosion function and therefore the erodibility of a material.

In coarse grained soils (sands and gravels), the critical shear stress has been empirically related to the mean grain size D_{50} (Briaud et al., 2001a).

$$\tau_c \text{ (N/m}^2\text{)} = D_{50} \text{ (mm)} \quad (12)$$

For such soils, the erosion rate beyond the critical shear stress is very rapid and one flood is long enough to reach the maximum scour depth. Therefore there is a need to be able to predict the critical shear stress to know if there will be scour or no scour but there is little need to define the erosion function beyond that point because the erosion rate is not sufficiently slow to warrant a time dependent analysis.

In fine grained soils (silts, clays) and rocks, equation 12 is not applicable (Figure 9.2) and the erosion rate is sufficiently slow that a time rate analysis is warranted. Therefore it is necessary to obtain the complete erosion function. An attempt was made to correlate those parameters, τ_c and S_i , to common soil properties in hope that simple equations could be developed for everyday use. The process consisted of measuring the erosion function on one hand and common soil properties on the other (water content, unit weight, plasticity index, percent passing sieve no. 200, undrained shear strength). This led to a database of 91 EFA tests (Table 9.5) which was used to perform regression analyses and obtain correlation equations (Figure 9.12 to 9.15). All attempts failed to reach a reasonable R^2 value.

The fact that no relationship could be found between the critical shear stress or the initial slope of the erosion function on one hand and common soil properties on the other seems to be at odds with the accepted idea that different cohesive soils erode at different rates. Indeed if different clays erode at different rates then the erosion function and therefore its parameters should be functions of the soils properties. The likely explanation is that there is a relationship between erodibility and soils properties but that this relationship is quite complicated, involves advanced soil properties, and has not been found. Instead, it was found much easier to develop an apparatus which could measure the erosion function on any sample of cohesive soil from a site. This apparatus was called the Erosion Function Apparatus or EFA.

9.8 The EFA: Erosion Function Apparatus

The EFA (Briaud et al. 1999, Briaud et al., 2001a) was conceived by Dr. Briaud in 1991, designed in 1992, and built in 1993 (Figure 9.16). The sample of soil, fine-grained or not, is taken in the field by pushing an ASTM standard Shelby tube with a 76.2 mm outside diameter (ASTM D1587). One end of the Shelby tube full of soil is placed through a circular opening in the bottom of a rectangular cross section pipe. A snug fit and an O-ring establish a leak proof connection. The cross section of the rectangular pipe is 101.6 mm by 50.8 mm. The pipe is 1.22 m long and has a flow straightener at one end. The water is driven through the pipe by a pump. A valve regulates the flow and a flow meter is used to measure the flow rate.

The range of mean flow velocities is 0.1 m/s to 6 m/s. The end of the Shelby tube is held flush with the bottom of the rectangular pipe. A piston at the bottom end of the sampling tube pushes the soil until it protrudes 1 mm into the rectangular pipe at the other end. This 1 mm protrusion of soil is eroded by the water flowing over it.

9.8.1 EFA test procedure

The procedure for the EFA test consists of

1. Place the sample in the EFA, fill the pipe with water, and wait one hour.
2. Set the velocity to 0.3 m/s.
3. Push the soil 1 mm into the flow.
4. Record how much time it takes for the 1 mm soil to erode (visual inspection)
5. When the 1 mm of soil is eroded or after 30 minutes of flow whichever comes first, increase the velocity to 0.6 m/s and bring the soil back to a 1 mm protrusion.
6. Repeat step 4.
7. Then repeat steps 5 and 6 for velocities equal to 1.0 m/s, 1.5 m/s, 2 m/s, 3 m/s, 4.5 m/s, and 6 m/s. The choice of velocity can be adjusted as needed.

9.8.2 EFA test data reduction

The test result consists of the erosion rate dz/dt versus shear stress τ curve (Figure 9.1, and 16). For each flow velocity v , the erosion rate dz/dt (mm/hr) is simply obtained by dividing the length of sample eroded by the time required to do so.

$$dz/dt = h/t \quad (13)$$

Where h is the length of soil sample eroded in a time t . The length h is 1 mm and the time t is the time required for the sample to be eroded flush with the bottom of the pipe (visual inspection through a Plexiglas window). After several attempts at measuring the shear stress τ in the apparatus it was found that the best way to obtain τ was by using the Moody Chart (Moody, 1944) for pipe flows.

$$\tau = f \rho v^2/8 \quad (14)$$

Where τ is the shear stress on the wall of the pipe, f is the friction factor obtained from Moody Chart (Figure 9.17), ρ is the mass density of water (1000 kg/m³), and v is the mean flow velocity in the pipe. The friction factor f is a function of the pipe Reynolds number Re and the pipe roughness ϵ/D . The Reynolds number is $Re = vD/\nu$ where D is the pipe diameter and ν is the kinematic viscosity of water (10⁻⁶ m²/s at 20°C). Since the pipe in the EFA has a rectangular cross section, D is taken as the hydraulic diameter $D = 4A/P$ (Munson et al., 1990) where A is the cross sectional flow area, P is the wetted perimeter, and the factor 4 is used to ensure that the hydraulic diameter is equal to the diameter for a circular pipe. For a rectangular cross section pipe:

$$D = 2ab/(a + b) \quad (15)$$

Where a and b are the dimensions of the sides of the rectangle. The relative roughness ϵ/D is the ratio of the average height of the roughness elements on the pipe surface over the pipe diameter D . The average height of the roughness elements ϵ is taken equal to $0.5D_{50}$ where D_{50} is the mean grain size for the soil. The factor 0.5 is used because it is assumed that the top half of the particle protrudes into the flow while the bottom half is buried into the soil

mass. During the test, it is possible for the soil surface to become rougher than just $0.5 D_{50}$; this occurs when the soil erodes block by block rather than particle by particle. In this case the value used for ε is estimated by the operator on the basis of inspection through the test window. Typical EFA test results are shown on Figure 9.1 for sand and then clay.

9.9 Some Existing Knowledge on Levee Erosion

9.9.1 Current Considerations in Design

The US Army Corps of Engineers' design manual (USACE, 2000) outlines the steps followed in the design and construction of levees (Table 9.6). The procedure does not include an evaluation of the erodibility of the soils used for the levees. A more in-depth discussion of design requirements is presented in Chapter 10.

9.9.2 Failure Mechanism

Flowing water exerts a tractive shear stress along the soil-water interface. The erosion process begins when this tractive shear stress exceeds the resistive force of the backslope soil (AlQaser, 1991). Hanson et al. (2003) describe four stages of erosion during the overtopping of cohesive embankments (Figure 9.18):

- Stage I:** Minor headcut movement up to the downstream embankment crest; surface erosion occurs.
- Stage II:** Headcut progresses from the downstream embankment crest to the upstream embankment crest.
- Stage III:** The crest lowers and breach formation begins as the headcut continues to migrate upstream of the embankment crest.
- Stage IV:** Erosion of the breach opening has progressed to near the base of the upstream toe of the embankment; driven by erosion of the sidewalls and development of an overhang, resulting in episodic mass failures and breach widening (Hunt et al., 2004).

Erosion typically occurs adjacent to some change or interruption in the flow pattern (Ralston, 1987). The turbulence associated with the flow disturbance breaks down the protective boundary laminar flow layer. This leads to the occurrence of full hydraulic stress intensity as well as rapid stress reversals, greatly increasing the erosion rate.

Gradually varied flow also leads to non-uniform erosion along the backslope producing overfalls. The overfall will advance progressively headward as long as the remaining embankment material can support the dam crest and upstream slope (Figure 9.19). The base of the overfall will deepen and widen.

As the eroding vertical overfall face advances headward, the overflow crest elevation will lower, cutting into the adverse grade of the upstream slope. This erosion pattern will continue and progress until the flow pattern changes into a free surface flow (Figure 9.20, AlQaser, Ruff, 1993). Headward advance of the overfalls is due to a combination of the following:

- 1) Insufficient soil strength to stand vertically due to the height of the face, stress relief cracking, and induced hydrostatic pressure in the cracks
- 2) Loss of foundation support for the vertical face due to the waterfall plunging effect and its associated lateral and vertical scour. As the vertical overfall gets higher, impact energy of the water fall increases, the rate of erosion increases and the scour hole becomes larger. The supporting foundation of the overfall face and sidewalls is thus removed.

The erosion pattern of embankments using non-cohesive soil is affected by the existence and location of a cohesive soil zone. For purely non-cohesive embankments, the erosion occurs on a uniform, but gradually flattening gradient. This erosion pattern can be modeled using the theory of tractive stress. The breach development is consistent with the principle of minimum rate of energy dissipation for streams (Coleman et al., 2002). Breaches, like streams, tend to alter their geometry in order to produce a minimum rate of energy dissipation. When the embankment includes a zone of cohesive soil, the overfall development will be retarded. If the zone is symmetrical, erosion will behave similar to that of a cohesive soil embankment. If the zone is an upstream sloping section, the overflow crest will degrade. This is due to undermining of the downstream non-cohesive zone. Portions of the overhanging cohesive zone will subsequently break off as the allowable bending moment is exceeded.

9.9.3 Numerical modeling

Erosion computer models are used to describe and quantify the complexities associated with an embankment breach. OVERFALL, a computer program developed by AIQaser (1991), predicts the heights and numbers of overfalls along the backslope of an overtopped embankment. Key features of breaches can also be reproduced with SIMBA, or SIMplified Breach Analysis (Temple et al., 2005). This model has been verified against embankment breach tests. Presently, SIMBA is only capable of addressing homogeneous embankment conditions. Future work will allow for applications to non-homogeneous field conditions, though.

Breach and discharge characteristics can be modeled and predicted with BREACH (Fread, 1988). BREACH allows for predictions of the size, shape, and time of formation of an earthen dam breach. A breach outflow hydrograph is also provided from the analysis. The extent of the enlargement, the peak outflow, and the time to peak flow are determined by the internal friction angle and the cohesive strength of the embankment soil. The BREACH model was verified by comparing the results of the model and several overtopping failure tests. These tests were conducted in different countries at varying scales with different homogeneous materials and construction practices. A summary of dam break numerical models that can be used for gradual failure is shown in Table 9.7.

9.9.4 Laboratory Tests

Nairn (1986) conducted two-dimensional flume tests to study cohesive shore erosion. Tests were conducted on artificial clay, composed of a bentonite-silt mixture, with and without an overlying veneer of sand. Surprisingly, the flume tests with sand did not lead to failure as the sand acted as an armor over the clay. Tests without sand, however, produced

responses close to those observed in the field. Table 9.8 displays the erosion rate results for the flume tests conducted by Nairn. Dodge (1988) also conducted laboratory flume tests to study erosion of a clayey sand (Figure 9.21). The results of the tests were not verified with field observations; they serve to provide a qualitative assessment of erosion.

AlQaser (1991) performed two laboratory tests to study progressive failure of an overtopped embankment (Figure 9.22). Both tests had the same design, but differed in the percent of sand in the soil. The first sample consisted of 80% clay and 20% sand. The other had 50% clay and 50% sand. Results show that the presence of more clay in the soil mixture leads to a greater vertical overfall height. The soil with more sand in the mixture, however, resulted in more horizontal overfall regression. It is concluded, therefore, that the physical and the geometrical properties of the embankment affect the number and heights of the developed overfalls.

9.9.5 Field Tests

Hanson, Cook, and Hahn (2001) describe preliminary evaluation of the headcut migration rates during overtopping and breaching tests on large-scale models. The headcut advance threshold was evaluated based on an energy dissipation term:

$$E = q\gamma_w H \quad (16)$$

Where q = unit discharge, γ_w = unit weight of water, and H = Headcut height. The headcut migration rates for each test section were evaluated and compared to measured soil properties, such as erodibility and soil strength. The results show that as soil strength decreases, the headcut migration rate increases (Figure 9.23).

The breaching of non-cohesive homogeneous embankments under constant-reservoir levels was studied using flume tests (Figure 9.24) by Coleman et al. (2002). This experiment simulated the failure of an embankment restricting a very large upstream reservoir. A small V breach was initiated and grew as erosion took place. A wide range of uniform non-cohesive soils were tested. The quantitative findings of these tests have not been verified by the results from large scale embankments.

It was found from the flume tests conducted by Coleman et al. (2002) that erosion progresses from primarily vertical to lateral in nature. This occurs as the breach channel invert approaches the foundation level. The channel invert slope will flatten as it rotates about a fixed pivot point, X_P , on the embankment (Figure 9.25).

The location of this pivot point is a function of the embankment sediment size. In plan view, the breach channel develops into an hourglass (or Venturi) shape (Figure 9.26). The curvature of the channel increases with time until the embankment foundation impedes the vertical erosion of the breach. This leads to an increase in the rounding of the approach and exit channels.

After the preliminary studies of 2001, Hanson, Cook, et al. (2003) performed a second study of the headcut migration and erosion widening rates during overtopping. They used large-scale models and three soils including two non-plastic (SM) silty sand materials and a

(CL) lean clay. The width of the breach during testing was evaluated using photographic measurements of the model embankment (Figure 9.27). Details of the testing indicated that headcut erosion was an important erosion process in the failure of cohesive embankments. It can influence the breach initiation time, breach formation time, breach width, peak discharge, and the overall outflow hydrograph.

The headcut migration rates (Figure 9.28), as well as the erosion widening rates (Figure 9.29), show a direct correlation to the compaction water content. The rate of breach widening was found by taking the linear regression of the breach width measurement from left bank to right bank versus time (Figure 9.30). The observed breach widths during testing were equal to two to five times the dam height. Figure 9.31 indicates that the head cutting rate for stages II and III of the erosion process is larger than the widening rate at the beginning but becomes approximately equal to it towards the end of the breaching process.

9.9.6 Factors Influencing Resistance to Overtopping

For a given soil, Hanson et al. (2003) show that erodibility correlates well with compaction water content, energy, density, and texture. By contrast, Cao et al. (2002) using a large data base found no relationship between common soil properties and the erodibility of cohesive soils. Dodge (1988, Figure 9.32) gave some trends of erodibility for cohesive soils using the plasticity chart. The FHWA (Chen, Cotton, 1988) also presents a plot of permissible shear stresses for cohesive soils based on the plasticity index (Figure 9.33).

Choliaras et al. (2003) concludes that the main measure of erosivity of overland flow is shear stress flow. He states that the increase of erosion rate is linear with shear stress of flow. He adds that for low values of surface shear stress, the erodibility of a soil decreases with increasing soil strength while for high values of surface shear stress, the erodibility of the soil increases with increasing surface strength.

According to Fread (1988), the growth of a breach is dependent on the soil properties of the dam. Unit weight, friction angle, and cohesive strength are shown to influence the size, shape, and time of formation of a breach. The amount of grass cover on the dam is also a factor in breach formation.

The results of the research performed by AlQaser (1991) point to poor compaction as a source of breaching. According to model tests conducted by Dodge (1988), the volume of scour produced during flow can be decreased by increasing the compaction of the soil.

Similarly, for clay soils, an increase in density reduces erodibility (Choliaras et al., 2003). For silty and sandy soils, the density or compaction of the soil does not significantly influence erodibility.

1953 Levee (Dike) failures in Netherlands

The Netherlands is a country of 8.5 Million people and 26% of them live below mean sea level protected by levees (Gerritsen, 2006). The following is a summary of an excellent article by Gerritsen in Geo-Strata (2006) which describes the 1953 disaster and the steps taken since

then by the Netherlands. Prior to 1953 the dikes were at a height equal to the maximum recorded water level plus 0.5 m. The height of some of the levees had been increased by constructing concrete walls along the levee crest. During World War II, the levees were used as a defense system and many holes were dug to that effect. After the war, the damage done to the levees was not adequately repaired.

On January 31, 1953, a North Sea storm combined with high tide and raised the water level to unprecedented height and 150 levee breaches occurred. During that storm, 1836 people died, 100,000 people evacuated, tens of thousands of livestock perished, and 136,500 hectares were inundated. The levee breaches were attributed to sustained wave overtopping. The land side of the levees was typically at a steeper slope (1v to 1.5h or 1v to 2h) than the sea side (1v to 3h or more). The failure process initiated from the land side and progressed backward towards the sea side. One sign of imminent failure was a longitudinal crack forming along the crest of the levee which was quickly filled by the rushing water.

On February 18, 1953, a committee was formed called the Delta Committee with the task of ensuring that such a disaster would not happen again. The committee chose to solve the problem not by increasing the height of the levees but rather by recommending the Delta Plan. This plan consisted of closing the shoreline completely through a series of permanent barriers to be built over a 20 year period. In 1975, due to political pressures from the fishing industry, the barriers were changed from complete damming to moveable storm surge barriers to be closed only in the event where a North Sea storm would coincide with a high tide.

The Netherlands now requires that the flood protection systems satisfy the following

- Be able to resist a storm surge with a probability of occurrence of 1/10,000 for the Province of Holland;
- Be able to resist a storm surge with a probability of occurrence of 1/4000 for less populated coastal areas; and
- Be reviewed and evaluated every 5 years with associated recommendations to be constructed in the following 5 years.

9.9.7 Influence of Grass Cover on Surface Erosion

Grass makes a difference in the resistance to surface erosion (Figure 9.34). The physical vegetative coverage on slopes provides increased resistance through underground soil reinforcement and surface protection (Li and Eddleman, 2002). Root systems aid slope stabilization through soil-root interaction. The mechanics of root-reinforcement are similar to the basic mechanics of engineering reinforced-earth systems (Coppin and Richards, 1990). The vegetation root growth reinforces the upper soil layers increasing the soil shear strength by over 33 % (Bhandari et al., 1998). Many researchers have developed theoretical models of root-reinforced soils, including Gray and Leiser (1982), Greenway (1987), Coppin and Richards (1990), Styczen and Morgan (1995), and Wu (1995). In general, the vegetative methods for surface erosion control include two types: herbaceous and woody. They all have the following four mechanisms in controlling surface erosion (Gray 1974; Greenway, 1987; Coppin and Richards, 1990):

1. Restraint: The root system binds the soil particles. The foliage residues restrain soil particle detachment via shallow, dense root systems, consequently reducing sediment transport.
2. Retardation: The foliage and stems increase the surface roughness and slow surface runoff.
3. Interception: The foliage and plant residues absorb the rainfall energy by intercepting the raindrops to reduce raindrop impacts.
4. Transpiration: Absorption of soil moisture by plants delays the initiation of saturation and increases shear strength by reducing pore-pressures.

The level of vegetation for protecting the soil depends on the combined effects of roots, stems and foliage (Coppin and Richards, 1990). Woody vegetation installed on slopes and streambanks provides resistance to shallow mass-movement by counterbalancing local instabilities. In order to achieve optimum stabilization, vegetation must establish quickly and solidly. For biotechnical stabilization techniques that only use vegetative materials, the stabilization is vulnerable at the early stage but becomes stronger as the vegetation is established (Li and Eddleman, 2002). For techniques that combine plant and inert materials such as dead wood, rocks or geosynthetics, inert materials support major loads at the early stage. As the vegetation matures, root systems will bind soils, inert materials and vegetation altogether on the slope or streambank, and increase the safety factor of structural protection (Biedenbarn et al., 1997).

From the engineering perspective, vegetation's use on slopes or streambanks may not be always ideal. Trees planted on certain parts of levees may have roots undermining the levee stability (USACE, 1999). Greenway (1987), and Coppin and Richards (1990) have analyzed vegetation's engineering functions and determined that its effects are both adverse and beneficial, depending on the circumstances. Therefore, selecting appropriate plant type becomes very critical in such conditions. This can be done by testing at large scale facilities such as the one at Texas A&M University which grows grass and tests it on slopes of various geometries.

Johnston (2003) prepared the chart of Figure 9.35 which gives the allowable shear stress at the interface between the soil and the water flowing on a slope. Different covers are represented including bare soil, grass covered, geosynthetic matting, hard armor. The depth D is the depth of water flowing over the slope S . Note that overall the range of slope covered is fairly shallow.

9.10 Soil and Water Samples Used for Erosion Tests

A total of 11 locations were identified for studying the erosion resistance of the levee soils. Emphasis was placed on levees which were very likely overtopped. These locations are labeled S1 through S15 for Site 1 through Site 15 on Figure 9.36. The samples were taken by pushing a Shelby tube when possible or using a shovel to retrieve soil samples into a plastic bag. For example at Site S1, the drilling rig was driven on top of the levee, stopped at the location of Site 1, a first Shelby tube was pushed with the drilling rig from 0 to 2 ft depth and then a second Shelby tube was pushed from 2 to 4 ft depth in the same hole. These two Shelby tubes belonged to boring B1. The drilling rig advanced a few feet and a second

location B2 at Site S1 was chosen; then two more Shelby tubes were collected in the same way as for B1. This process at Site S1 generated 4 Shelby tube samples designated

- S1-B1-(0-2ft)
- S1-B1-(2-4ft)
- S1-B2-(0-2ft)
- S1-B2-(2-4ft)

Four such Shelby tubes were collected from sites S1, S2, S3, S7, S8, and S12. In a number of cases, Shelby tube samples could not be obtained because access for the drilling rig was not possible (e.g.: access by light boat for the MRGO levee) or pushing a Shelby tube did not yield any sample (clean sands). In these cases, grab samples were collected by using a shovel and filling a plastic bag. The number of bags collected varied from 1 to 4. Plastic bag samples were collected from sites S4, S5, S6, S11, and S15. The total number of sites sampled for erosion testing was therefore 11. These 11 sites generated a total of 23 samples. One of the samples, S8-B1-(2-4ft), exhibited two distinct layers during the EFA tests and therefore lead to two EFA curves. All in all 24 EFA curves were obtained from these 23 samples: 14 performed on Shelby tube samples and 10 on bag samples. The reconstitution of the bag samples in the EFA is discussed later.

Water salinity has an effect on erosion. The salinity of the water was determined by using the soil samples collected at the sites. Samples S11 and S15 were selected because one was on the Lake Pontchartrain side and the other on the Lake Borgne side. The procedure to obtain theconsisted of:

1. Dry the soil (about 70 g) in an oven for 12 hr
2. Weigh a quantity of soil, e.g. 10 g and place it in a PE bottle
3. Add deionized (DI) water in the ratio of 2 ml water for one sample and 5 ml water for another sample to each gram of soil
4. Soil: DI water = 10 g: 20 ml or 10g: 50 ml
5. Shake the bottle to thoroughly mix the soil and water
6. Allow the soil to settle for 12 hr
7. Use a pH meter (Orion model 420 A) to measure the pH and a calibrated conductivity meter (Corning model 441) to measure the conductivity of the water.
8. Perform a calibration of the conductivity meter by using known concentrations of salt.
9. Use the conductivity to salinity calibration curve to obtain the salinity of the water created in steps 1 to 7.

Then it becomes necessary to correct the salinity of this water because the amount of water added to the soil for the salinity determination test does not correspond to the amount of water available in the soil pores in its natural state (in the levee). This is done by calculating the amount of water available in the pores of the samples in its natural state. This requires the use of the void ratio and the degree of saturation of the samples calculated using simple phase diagram relationships. The results obtained are shown in Table 9.9.

9.11 Erosion Function Apparatus (EFA) Test Results

9.11.1 Sample Preparation

No special sample preparation was necessary for the samples which were in Shelby tubes. The Shelby tube was simply inserted in the hole on the bottom side of the rectangular cross section pipe of the FEA (described previously).

For bag samples obtained by using a shovel to collect the soil, there was a need to reconstruct the sample. These samples were prepared by re-compacting the soil in the Shelby tube (Figure 9.37). The same process as the one used to prepare a sample for a Proctor compaction test was used. Since it was not known what the compaction level was in the field, two extreme levels of compaction energy were used to recompact the samples. The goal was to bracket the erosion response of the intact soil.

For the high compaction effort (100% of Modified Proctor compaction effort), the sample was compacted in an 18-inch long Shelby tube as follows:

- 1) The total sample height was 6 inches. The sample was compacted in eight layers.
- 2) To form each layer, the soil was poured into the Shelby tube from a height of 1 inch above the top of the tube.
- 3) The soil was compacted using a 10 lb hammer (Modified Proctor hammer) with a drop height of 1.5 feet. Each layer was compacted by 8 hammer blows, i.e. 8 blows/layer.
- 4) This process was repeated until a 6 inch sample was obtained.
- 5) The corresponding compaction energy was equal to the Standard Modified Proctor Compaction energy.

For the low compaction effort (1.63% of Modified Proctor compaction effort), the sample was compacted in an 18-inch long Shelby tube as follows.

1. The total sample height was 6 inches. The sample was compacted in eight layers.
2. To form each layer, the soil was poured into the Shelby tube from a height of 1 inch above the top of the tube.
3. The soil was compacted using a 10 lb hammer (Modified Proctor hammer) with a drop height of 1 inch. Each layer was compacted by 3 hammer blows, i.e. 3 blows/layer.
4. This process was repeated until a 6 inch sample was obtained.
5. The corresponding compaction energy was 1.63% of the Standard Modified Proctor Compaction energy.

9.11.2 Sample EFA Test Results

The procedure described earlier was strictly followed for the EFA tests. The results were prepared in the form of a word file report and an accompanying excel spread sheet detailing the data reduction and associated calculations. The main result of an EFA test is a couple of plots: one is the plot of the erosion rate versus mean velocity in the EFA pipe, the other is the plot of the erosion rate versus shear stress at the interface between the soil and the

water. These two plots are collected in Appendix A for all 24 EFA tests. Figs. 9.38 and 9.39 show two examples of results for a very erodible soil and a very erosion resistant soil.

9.11.3 Summary Erosion Chart

In an effort to give a global rendition of the EFA results, an erosion chart was created. The blank erosion chart has been presented earlier and is reproduced here for convenience (Figure 9.40). This chart allows one to present the erosion curves in a way which categorizes the soils according to one erosion category. Category 1 is very erodible and refers to soils such as clean fine sands. Category 5 is very erosion resistant and refers to soils such as some of the highly compacted and well graded clays.

Figure 9.41 shows the erosion chart populated with the EFA results for all 24 EFA tests. The legend contains the sample/test designation which starts with the site number (Figure 9.36), followed by the boring number, the depth, and letter symbols including SW, TW, LC, HC, and LT. SW stands for Sea Water and means that the water used in the EFA test was salt water at a salinity of approximately 35000 ppm. TW stands for Tap Water and means that the water used in the EFA test was Tap Water at a salinity of approximately 500 ppm. LC stands for Low Compaction, refers to bag samples only, and means that the sample was prepared using 1.6% of Modified Proctor compaction effort. HC stands for High Compaction, refers to bag samples only, and means that the sample was prepared using 100% of Modified Proctor compaction effort. LT stands for Light Tamping and refers to the preparation of some bag samples used in some early tests; it is very similar to the LC preparation.

One of the first observations coming from the summary erosion chart on Figure 9.41 is that the erodibility of the soils obtained from the New Orleans levees varies widely all the way from very high erodibility (Category 1) to very low erodibility (Category 5). This explains in part why some of the overtopped levees failed while other overtopped levees did not. This finding points to the need to evaluate the remaining levees for erodible soils (weak links).

9.11.4 Influence of Compaction on Erodibility

Several of the bag samples were tested at two extreme compaction efforts: 100% Modified Proctor and 1.6% Modified Proctor. Because the low and high compaction samples originated from the same bag of collected soil, it is reasonable to assume that the samples are very similar. The EFA tests results aimed at identifying the influence of the compaction effort are isolated in Figure 9.42. Sample S4 shows a major influence of the compaction effort on the erodibility. Indeed, the low compaction sample is at the border between Category 1 and Category 2 (very high to high erodibility) while the high compaction sample is at the border between Category 4 and Category 5 (very low to low erodibility). However, Samples S15 and S11 do not show much difference between the high compaction and the low compaction.

The index properties of the samples tested are presented in a following section. Sample S4 is a high plasticity silt. It has 90.47 % fines, a plasticity index of 30, and a USCS classification of MH. Sample S11 is a clean uniform sand. It has 0.1 % fines, and a USCS classification of SP. Sample S15 is a silty sand. It has 29.89 % fines, and a USCS classification of SM. These three data points tend to indicate that compaction has a more

significant influence on erodibility for some soils (higher fines content) than for others (lower fine content).

9.11.5 Influence of water salinity on erodibility

Salinity can have an influence on the erodibility of a soil. Several of the samples were tested by using water at two extreme salt concentrations: 35000 ppm to simulate sea water and 500 ppm to simulate water with a very low salt concentration. Because the samples used to check the influence of the water salinity originated from different Shelby tubes at two different depths (0-2 ft and 2-4 ft), it is possible that the samples may have had different erodibility to start with. This may have clouded the influence of the water salinity.

The EFA tests results for the tests aimed at identifying the influence of the water salinity are isolated in Figure 9.43. Conclusions are difficult to draw because the samples may not be from the same soil. One sample (S8-B1) actually was made of two separate layers which had two different erosion functions and lead to two EFA curves for the same Shelby tube.

Nevertheless, the following observations can be made. Samples S12 show that an increase in water salinity increases the resistance to erosion, samples S2 and S8 show no influence, while samples S1 and S7 show a reverse influence of the water salinity. The index properties of the samples tested are presented in a following section. All samples exhibit a high clay content.

9.12 Index Properties of the Samples Tested in the EFA

A set of index property tests were performed on the samples used in the EFA. Some of the tests were performed by Soil Testing Engineering in Baton Rouge, the remainder of the tests were performed at Texas A&M University. Table 9.10 shows a summary of the results as well as the classifications according the Unified Soil Classification System. As can be seen there are no gravels, and mostly sands, silts, and clays.

9.13 Levee Overtopping and Erosion Failure Guideline Chart

In an effort to correlate the results of the EFA erosion tests with the behavior of the levees during overtopping flow, Figure 9.44 was prepared. It seems reasonably sure that the levees at sites S4, S5, S6, and S15 were overtopped and failed. At the same time it seems reasonably sure that the levees at sites S2, and S3 were overtopped and resisted remarkably well. The dark circles on Figure 9.44 correspond to samples taken from levees that were overtopped and failed by erosion while the open circles correspond to samples taken from levees that were overtopped and held up well during that overtopping.

Figure 9.44 shows a definite correlation between the EFA tests results and the behavior of the levees during overtopping. Figure 9.45 was generated from Figure 9.44 as a levee guideline for erosion resistance during overtopping. It is suggested that such EFA erosion tests should be used in the future to predict levee behavior and ensure erosion resistance to overtopping. In addition, this type of testing can be performed on an increased variety of soils, and with varied compaction conditions, to develop generalized relationships

between soil types and soil characteristics, placement and compaction conditions, and resistance to erosion.

9.14 Summary

The results of this pilot study show that we are well able to correlate soil type, soil characteristics, and placement and compaction conditions with embankment performance with regard to erodibility. Moreover, we are also able to perform erodibility tests of specific soils, and for specific placement and compaction conditions, and the results of these tests appear to correlate well with observed field performance with regard to erodibility during levee overtopping in the New Orleans region during hurricane Katrina.

Accordingly, it is clearly possible to identify and avoid the use of materials that can be expected to perform poorly with regard to erosion resistance, and it is feasible to design engineered embankments with a high intrinsic resistance to erosion. That would have been very useful in the New Orleans regional flood protection system, and it appears that avoiding the use of highly erodeable levee embankment fills, and using instead embankment fills engineered to provide improved erosion resistance, would likely have significantly reduced both damages and loss of life in this event.

There is more to be done in further developing and refining these testing procedures and developing generalized correlations between material characteristics and placement conditions vs. erodibility, and also with regard to the development of corollary analysis methods and procedures for making engineering assessments regarding likely rates of erosion, etc., as a function of overtopping intensities, geometries and durations. Nonetheless, it appears that we are able to include resistance to erosion as a deliberately engineered feature of levee embankments. As this adds a potentially important source of additional system resilience, this should be considered in the future for flood protection systems defending large populations at risk.

9.15 References

- AlQaser, G., and Ruff, J. F., (1993), "Progressive failure of an overtopped embankment," Proceedings of the 1993 National Conference on Hydraulic Engineering, Hydraulic Division of ASCE, pp. 1957-1962.
- AlQaser, G.N., (1991), Progressive Failure of an Overtopped Embankment, Ph.D. dissertation, Colorado State University, Fort Collins, CO.
- Apperly, L.W., 1968, "Effect of turbulence on sediment entrainment", PhD dissertation, University of Auckland, Auckland, New Zealand.
- Ariathurai, R., and Arulanandan, K. (1978). "Erosion rates of cohesive soils." J. Hydr. Div., ASCE, 104(2), 279–283.
- Arulanandan, K. (1975). "Fundamental aspects of erosion in cohesive soils." J. Hydr. Div., ASCE, 101(5), 635–639.

- Arulanandan, K., Loganathan, P., and Krone, R. B. (1975). "Pore and eroding fluid influence on surface erosion of soil." *J. Geotech. Engrg. Div., ASCE*, 101(1), 51–66.
- ASTM D1587, American Society for Testing and Materials, Philadelphia, USA.
- Bhandari, G., Sarkar, S. S., and Rao, G. V. (1998). *Erosion control with geosynthetics, Geohorizon: State of art in geosynthetic technology*, A.A. Balkema, Rotterdam, Netherlands.
- Biedenharn, D. S., Elliott, C. M., and Watson, C. C. (1997). *The WES Stream Investigation and Streambank Stabilization Handbook*. US Army Engineer Waterways Experiment Station, Vicksburg, MS.
- Briaud J.-L., Ting F., Chen H.C., Cao Y., Han S.-W., Kwak K., "Erosion Function Apparatus for Scour Rate Predictions". *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol.127, No. 2, 2001a, pp.105-113.
- Briaud J.-L., Ting F., Chen H.-C., Gudavalli R., Kwak K., Philogene B., Han S.-W., Perugu S., Wei G.S., Nurtjahyo P., Cao Y.W., Li Y., "SRICOS: Prediction of Scour Rate at Bridge Piers, TTI Report no. 2937-1 to the Texas DOT, 1999, Texas A&M University, College Station, Texas, USA.
- Briaud, J. L., Chen, H. C. Kwak K., Han S-W., Ting F., "Multiflood and Multilayer Method for Scour Rate Prediction at Bridge Piers", *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol.127, No. 2, 2001b, pp.105-113.
- Briaud, J.-L., Ting, F., Chen, H.C., Gudavalli, S.R., Perugu, S., and Wei, G., "SRICOS: Prediction of Scour Rate in Cohesive Soils at Bridge Piers", *ASCE Journal of Geotechnical Engineering*, Vol.125, 1999, pp. 237-246.
- Cao Y., Wang J., Briaud J.L., Chen H.C., Li Y. Nurtjahyo P., 2002, "EFA Tests and the influence of Various Factors on the Erodibility of Cohesive Soils", *Proceedings of the First International Conference on Scour of Foundations*, Texas A&M University, Dpt. of Civil Engineering, College Station.
- Cao, Y., "The Influence of Certain Factors on the Erosion Functions of Cohesive Soil", *Master Thesis*, 2001, Texas A&M University, College Station, Texas, USA
- Chen, Y.H., Cotton, G.K., (1988), "Design of Roadside Channels with Flexible Linings," *Federal Highway Administration, Hydraulic Engineering Circular No. 15*.
- Choliaras, I.G., Tantos, V.A., Ntalos, G.A., Metaza, X.A., (2003), "The influence of soil conditions on the resistance of cohesive soils against erosion by overland flow," *Journal of International Research Publication*, Issue 3.
- Christensen, B. A. (1965). "Discussion of 'Erosion and deposition of cohesive soils,' by E. Partheniades." *J. Hydr. Div., ASCE*, 91(5), 301– 308.

- Coleman S.E., Andrews D.P., Webby M.G., (2002), "Overtopping Breaching of Non Cohesive Homogeneous Embankments", Journal of Hydraulic Engineering, Vol. 138, No. 9, ASCE, Reston, Virginia, USA.
- Coppin, N. J., and Richards, I. G. (1990). Use of Vegetation in Civil Engineering. Butterworths, London.
- Dodge, R.A., (1988), Overtopping Flow on Low Embankment Dams – Summary Report of Model Tests, REC-ERC-88-3, U.S. Bureau of Reclamation, Denver, Colorado.
- Dunn, I. S. (1959). "Tractive resistance to cohesive channels." J. Soil Mech. and Found. Div., ASCE, 85(3), 1–24.
- Einstein, H.A., El-Samni, E.S., 1949, "Hydrodynamic forces on a rough wall", Review of modern physics, 21, 520-524.
- Enger, P. F., Smerdon, E. T., and Masch, F. D. (1968). "Erosion of cohesive soils." J. Hydr. Div., ASCE, 94(4), 1017–1049.
- Fread, D.L., (1988), BREACH: An Erosion Model for Earthen Dam Failures, National Weather Service, National Oceanic and Atmospheric Administration, Silver Spring, Maryland.
- Gerritsen, H. (2006), "The 1953 Dike Failures in the Netherlands, Geo-Strata, March-April 2006, p18-21, Geo-Institute of the American Society of Civil Engineers, Reston, Virginia, USA.
- Gray, D. H. (1974). "Reinforcement and stabilization of soil by vegetation." J. Geotech. Engr. Div., ASCE, 100(GT6), 695-699.
- Gray, D. H., and Leiser, A. T. (1982). Biotechnical Slope Protection and Erosion Control. Van Nostrand Reinhold, New York.
- Gray, D. H., and Sotir, R. B. (1996). Biotechnical and Soil Bioengineering Slope Stabilization: a Practical Guide for Erosion Control. John Wiley & Sons, Inc., New York.
- Greenway, D. R. (1987). Vegetation and slope stability. In: M.G. Anderson, K.S. Richards (Eds.). Slope Stability: Geotechnical Engineering and Geomorphology, John Wiley & Sons, New York, pp. 187-230.
- Hanson, G.J., Cook, K.R., (2004), "Apparatus, test procedures, and analytical methods to measure soil erodibility in situ," Applied Engineering in Agriculture, American Society of Agricultural Engineers, Vol. 20, No. 4, pp. 455-462.
- Hanson, G.J., Cook, K.R., Hahn IV, W., (2001), "Evaluating headcut migration rates of earthen embankment breach tests," Written for Presentation at the 2001 ASAE Annual International Meeting, Paper No. 01-012080, Sacramento, California.

- Hanson, G.J., Cook, K.R., Hahn IV, W., Britton, S.L., (2003), "Evaluating erosion widening and headcut migration rates for embankment overtopping tests", Written for Presentation at the 2003 ASAE Annual International Meeting, Paper No. 032067, Las Vegas, Nevada.
- Hanson, G.J., Cook, K.R., Hahn IV, W., Britton, S.L., (2003), "Observed Erosion Processes During Embankment Overtopping Tests," Written for presentation at the 2003 ASAE Annual International Meeting, Paper No. 032066, Las Vegas, Nevada.
- Hanson, G.J., Cook, K.R., Hunt, S., (2005), "Physical modeling of overtopping erosion and breach formation of cohesive embankment," Transactions of the ASABE, Vol. 48, No. 5, pp. 1783-1794.
- Hanson, G.J., Simon, A., (2001), "Erodibility of cohesive streambeds in the loess area of the Midwestern USA", Hydrological Processes, Vol. 15, pp. 23–38.
- Hanson, G.J., Temple, D.M., Morris, M., Hassan, M., Cook, K., (2005), "Simplified Breach Analysis Model For Homogeneous Embankments: Part II, Parameter Inputs And Variable Scale Model Comparisons," Proceedings Of 2005 U.S. Society On Dams Annual Meeting And Conference, Salt Lake City, Utah, pp. 163-174.
- Hunt, S.L., Hanson, G.J., Cook, K.R., Kadavy, K.C., (2004), "Breach Widening Observations from Earthen Embankment Tests," American Society of Agricultural Engineers/Canadian Society for Engineering Annual International Meeting, Ottawa, Canada, Paper No. 042080.
- Johnston A. (2003), "Range of Allowable Shear Stresses by Depth and Slope", personal communication.
- Kelly, E. K., and Gularte, R. C. (1981). "Erosion resistance of cohesive soils." J. Hydr. Div., ASCE, 107(10), 1211–1224.
- Li, M.-H., and Eddleman, K. E. (2002). "Biotechnical engineering as an alternative to traditional engineering methods: a biotechnical streambank stabilization design approach." Landscape and Urban Planning 60(4), 225-242.
- Lyle, W. M., and Smerdon, E. T. (1965). "Relation of compaction and other soil properties to erosion and resistance of soils." Trans., ASAE, American Society of Agricultural Engineers, St. Joseph, Mich., 8(3).
- Mitchell, J.K., 1993, "Fundamentals of Soil Behavior", 2nd Ed., John Wiley and Sons, New York.
- Moody L.F., "Friction Factors for Pipe Flow", Transaction of the American Society of Civil Engineers, 1944, Vol. 66, Reston, Virginia, USA.
- Munson, B. R., Young, D. F., and Okiishi, T. H. (1990). Fundamentals of fluid mechanics. Wiley, New York.

- Nairn, R.B., (1986), "Physical Modeling of Wave Erosion on Cohesive Profiles," Proceedings, Symposium on Cohesive Shores, Burlington, Ontario.
- Ralston, D.C., (1987), "Mechanics of embankment erosion during overflow," Proceedings of the 1987 National Conference on Hydraulic Engineering, Hydraulics Division of ASCE, pp. 733-738.
- Richardson, E. V., and Davis, S. R. (1995). "Evaluating scour at bridges." Rep. No. FHWA-IP-90-017 (HEC 18), Federal Highway Administration, Washington, D.C.
- Shaikh, A., Ruff, J. F., and Abt, S. R. (1988). "Erosion rate of compacted Na-montmorillonite soils." J. Geotech. Engrg., ASCE, 114(3), 296– 305.
- Shields, A. (1936). "Anwendung der Aehnlichkeits-Mechanik und der Turbulenzforschung auf die Geschiebebewegung." Preussische Versuchsanstalt für Wasserbau und Schiffbau, Berlin (in German).
- Shields, A., "Application of similarity principles, and turbulence research to bed-load movement," 1936, California Institute of Technology, Pasadena (translated from German).
- Smerdon, E. T., and Beasley, R. P. (1959). "Tractive force theory applied to stability of open channels in cohesive soils." Res. Bull. No. 715, Agricultural Experiment Station, University of Missouri, Columbia.
- Styczen, M. E., and Morgan, R. P. C. (1995). Engineering properties of vegetation. In: R.P.C. Morgan, R.J. Rickson (Eds.). Slope Stabilization and Erosion Control: A Bioengineering Approach. E & FN Spon, London, pp. 5-58.
- Temple, D.M., Hanson, G.J., Neilsen, M.L., Cook, K.R., (2005), "Simplified Breach Analysis Model For Homogeneous Embankments: Part I, Background And Model Components," Proceedings Of The 2005 U.S. Society On Dams Annual Meeting And Conference, Salt Lake City, Utah, pp. 151-161.
- USACE (1999). Guidelines for Landscape Planting and Vegetation Management at Floodways, Levees, and Embankment Dams. Engineering Manual 1110-2-301, U.S. Army Corps of Engineers, Washington, D.C.
- USACE, (2000), "Engineering and Design – Design and Construction of Levees," Publication No. EM 1110-2-1913.
- Vanoni, V. A., "Sedimentation Engineering", ASCE-Manuals and Reports on Engineering Practice no. 54, 1975, prepared by the ASCE task committee.
- White, C.M., 1940, "The Equilibrium of grains on the bed of a stream", Proc. Royal Soc. of London, 174(958), 322-338.
- Wu, T.H. (1995). Slope stabilization. In: R.P.C. Morgan, R.J. Rickson (Eds.). Slope Stabilization and Erosion Control: A Bioengineering Approach. E & FN Spon, London, pp. 221-264.

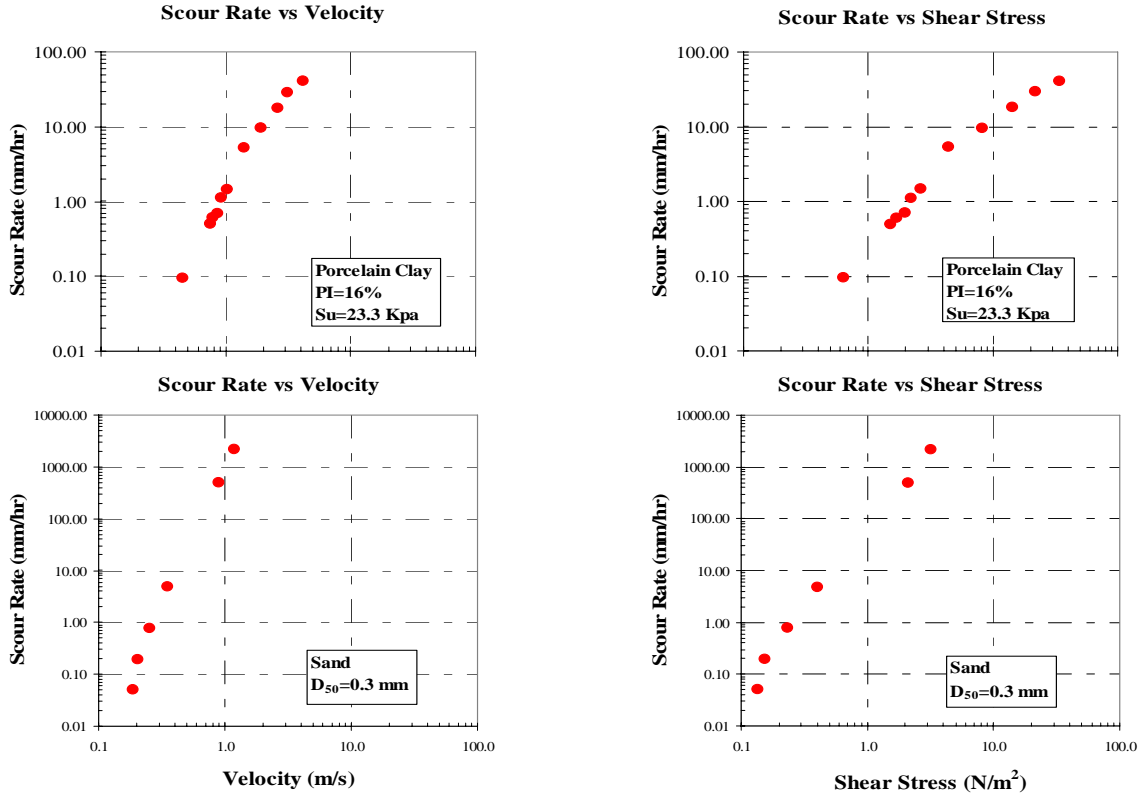


Figure 9.1: Erodibility function for a clay and for a sand.

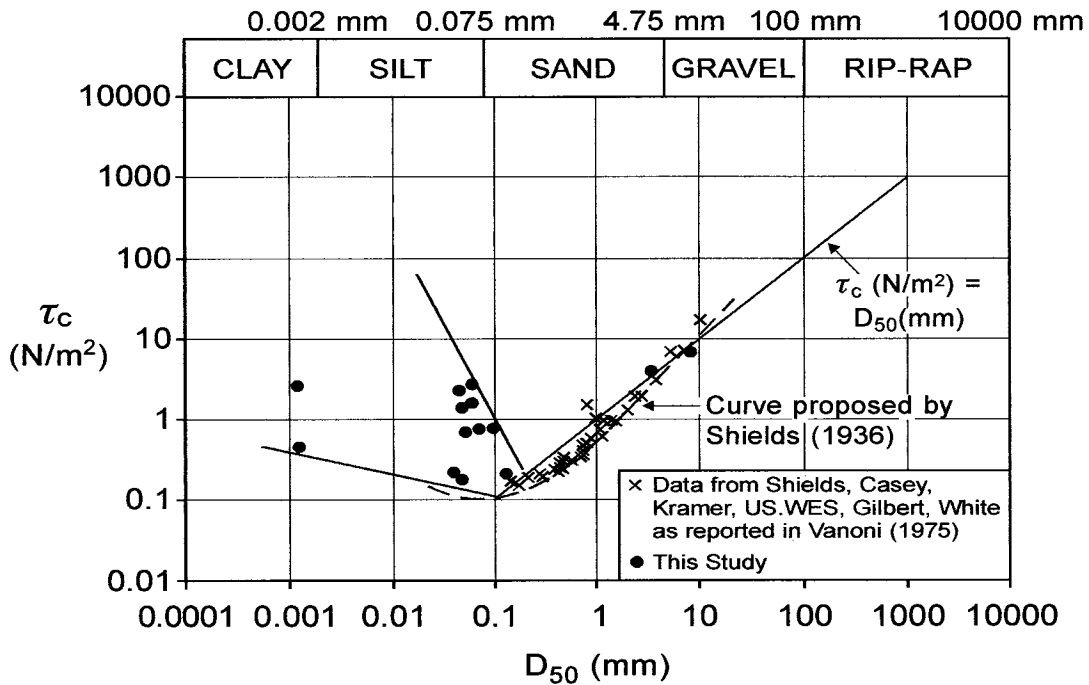


Figure 9.2: Critical shear stress versus mean soil grain size.

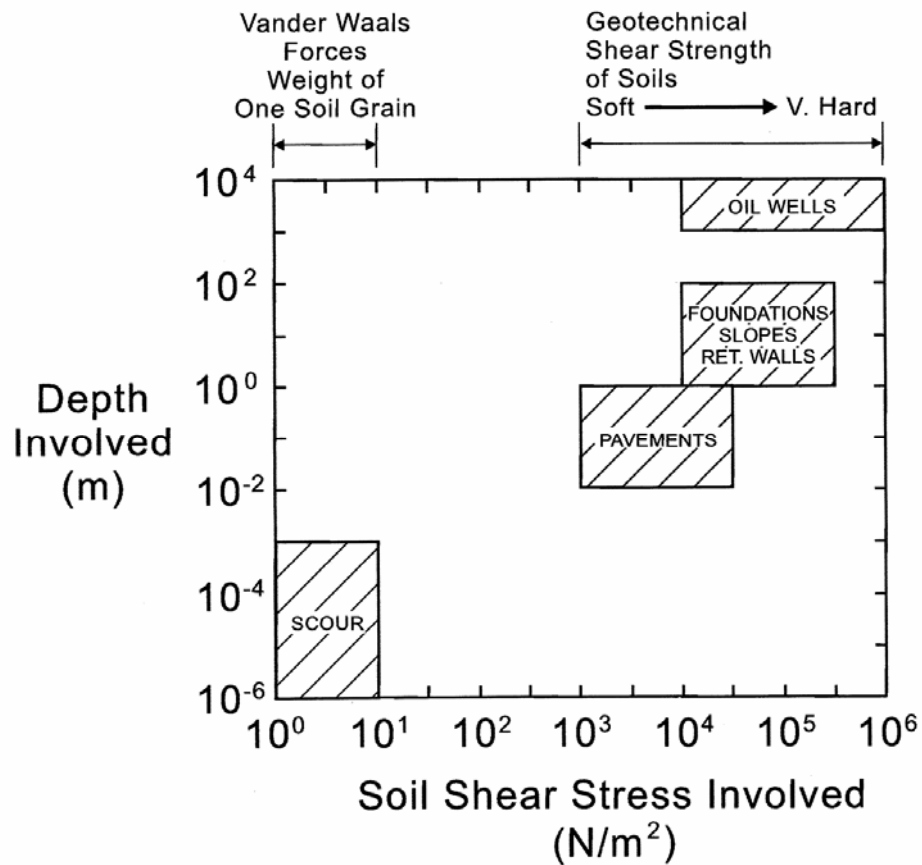


Figure 9.3: Magnitude of shear stresses involved in various fields of engineering.

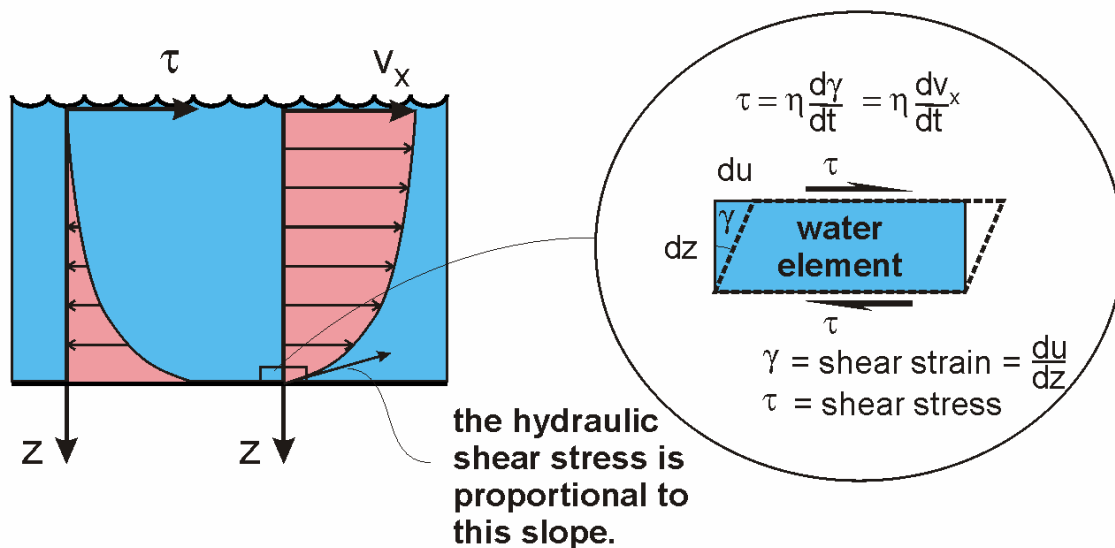


Figure 9.4: Velocity and shear stress within the flow depth.

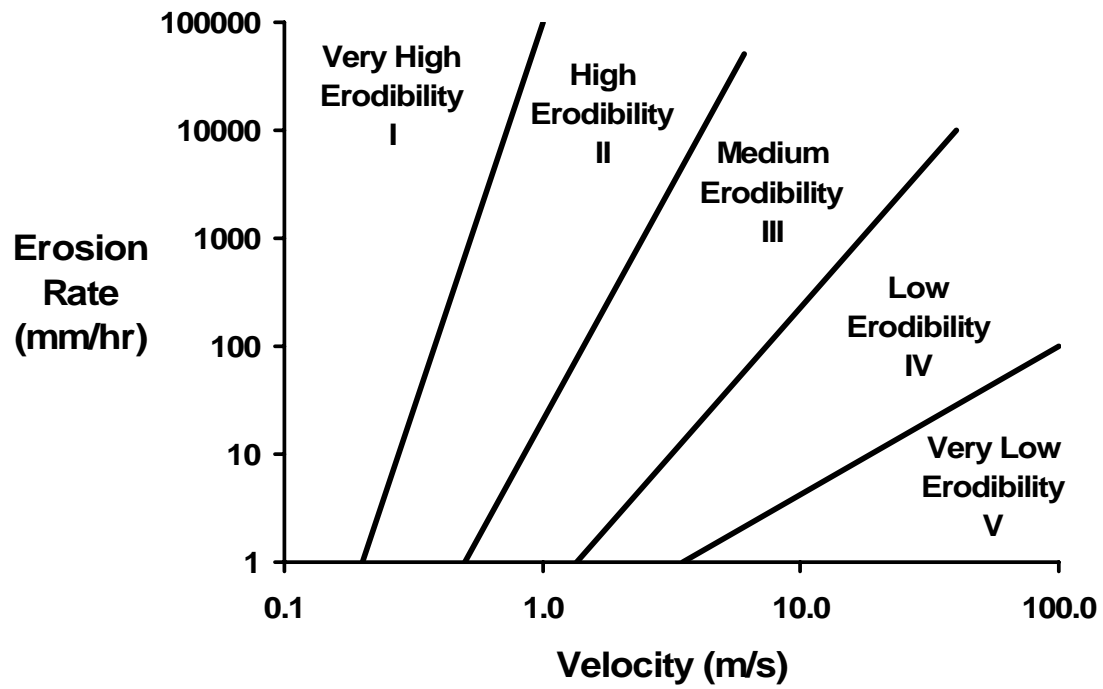


Figure 9.5: Erosion Categories.

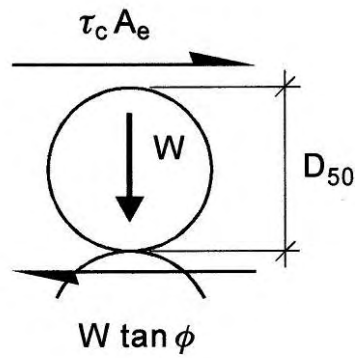


Figure 9.6: Particle diagram for a simple sliding mechanism.

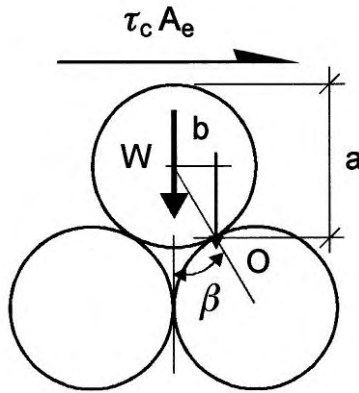


Figure 9.7: Particle diagram for a simple rolling mechanism.

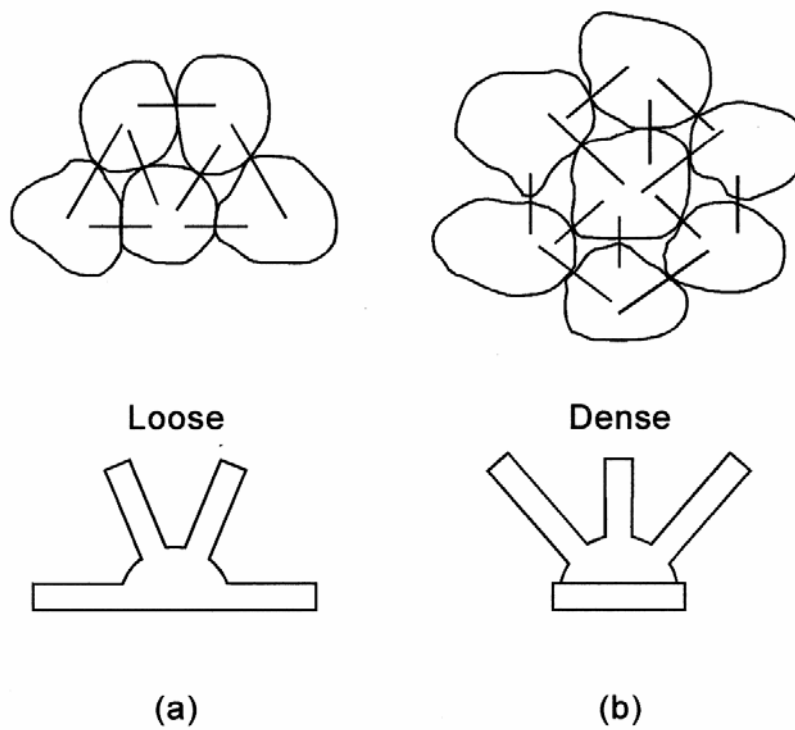


Figure 9.8: Contact angle distributions in coarse grained soils.

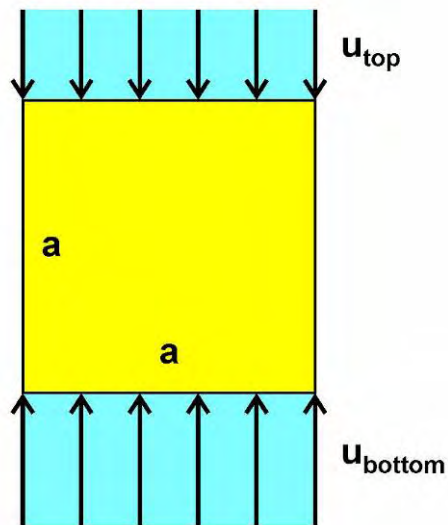


Figure 9.9: Particle diagram for a simple plucking mechanism.

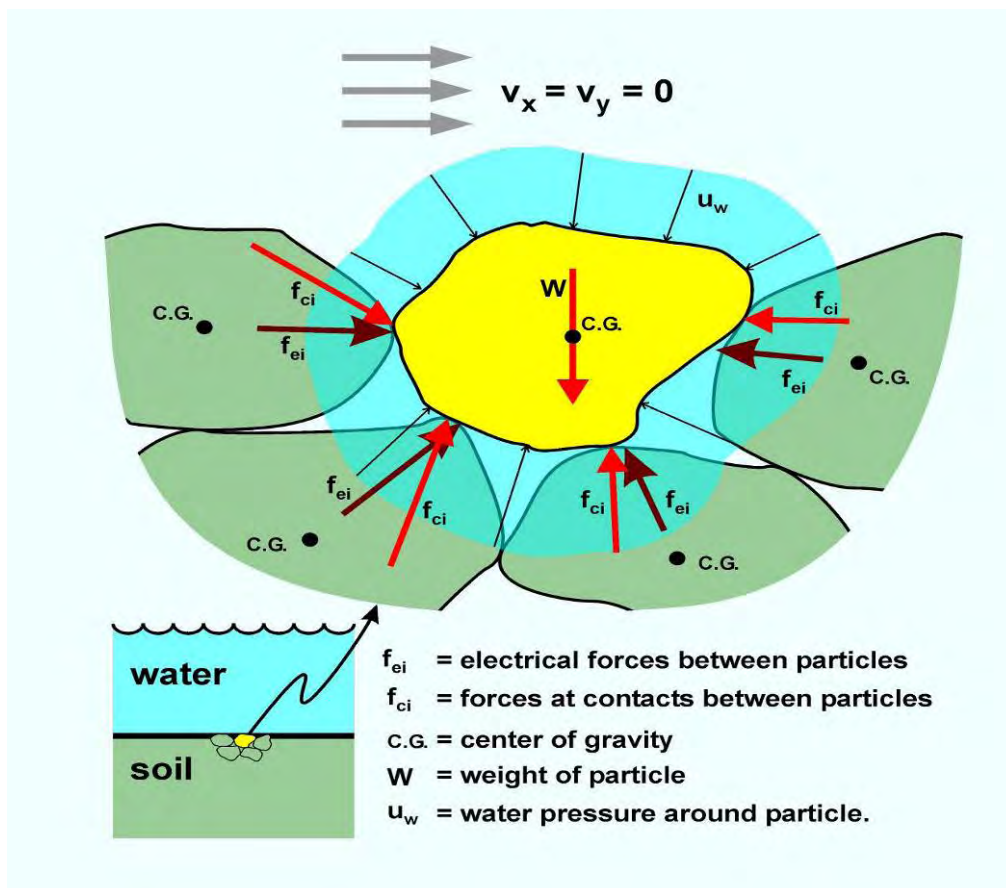


Figure 9.10: Forces and pressure on particle: no flow condition

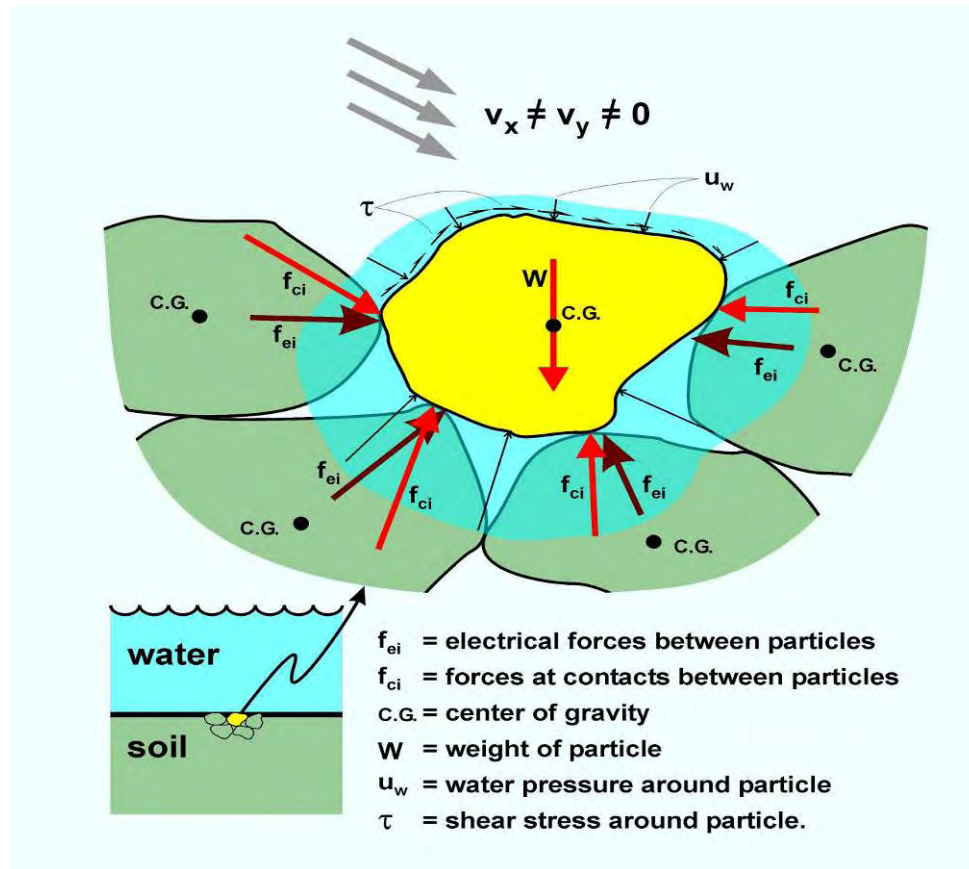


Figure 9.11: Forces and pressure on particle: flow condition.

Table 9.1: Gravity and Van der Waals Forces for Sand and Clay Particles

	Sand particle	Clay particle
Diameter d (m)	2×10^{-3}	1×10^{-6}
Weight W (N)	1.1×10^{-3}	1.36×10^{-13}
Van der Waals attraction F_{VDW} (N)	7.85×10^{-23}	3.14×10^{-16}
F_{VDW}/W	7.1×10^{-20}	2.3×10^{-3}

Table 9.2: Measured Critical Shear Stress in Clays

Authors	Range of τ_c (N/m ²)
Dunn (1959)	2–25
Enger et al. (1968)	15–100
Hydrotechnical Construction, Moscow (1936)	1–20
Lyle and Smerdon (1965)	0.35–2.25
Smerdon and Beasley (1959)	0.75–5
Arulanandan et al. (1975)	0.1–4
Arulanandan (1975)	0.2–2.7
Kelly and Gularte (1981)	0.02–0.4

Table 9.3: Measured Erosion Rates in Clay

Authors	Results	Inferred scour rate (mm/hr)*
Richardson, Davis (1995)	Maximum scour depth reached in days	10-100
Arulanandan et al. (1975)	1-4 g/cm ² /min	300-1200
Shaikh et al. (1988)	0.3-0.8 N/m ² /min	9-24
Ariathurai, Arulanandan (1978)	0.005-0.09 g/cm ² /min	1.5-27
Kelly, Gularte (1981)	0.0057-0.01 g/cm ² /s	100-180

* Erosion rate $dz/dt = (\text{weight loss rate per unit area } dw/a \text{ dt})/(\text{unit weight } \gamma)$

Table 9.4: Factors Influencing the Erodibility of Fine Grained Soils

<u>When this parameter increases</u>	<u>Erodibility</u>
Soil water content	
Soil unit weight	decreases
Soil plasticity Index	decreases
Soil undrained shear strength	increases
Soil void ratio	increases
Soil swell	increases
Soil mean grain size	
Soil percent passing sieve #200	decreases
Soil clay minerals	
Soil dispersion ratio	increases
Soil cation exchange capacity	
Soil sodium absorption ratio	increases
Soil pH	
Soil temperature	increases
Water temperature	increases
Water chemical composition	

Table 9.5: Database of EFA tests

Woodrow Wilson Bridge (Washington)	Tests 1 to 12
South Carolina Bridge	Tests 13 to 16
National Geotechnical Experimentation Site (Texas)	Tests 17 to 26
Arizona Bridge (NTSB)	Test 27
Indonesia samples	Tests 28 to 33
Porcelain clay (man-made)	Tests 34 to 72
Bedias Creek Bridge (Texas)	Tests 73 to 77
Sims Bayou (Texas)	Tests 78 to 80
Brazos River Bridge (Texas)	Test 81
Navasota River Bridge (Texas)	Tests 82 and 83
San Marcos River Bridge (Texas)	Tests 84 to 86
San Jacinto River Bridge (Texas)	Tests 87 to 89
Trinity River Bridge (Texas)	Tests 90 and 91

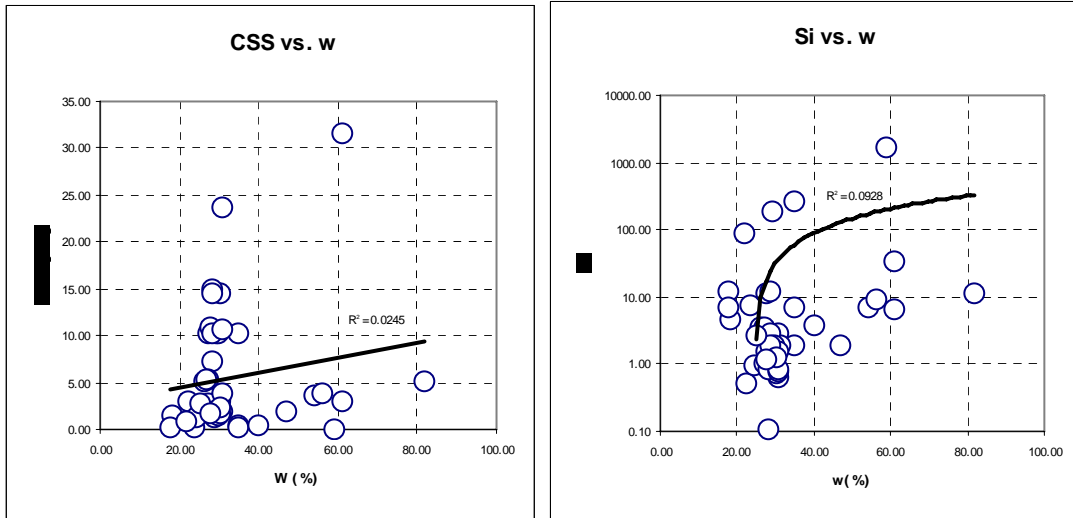


Figure 9.12: Erosion properties as a function of water content.

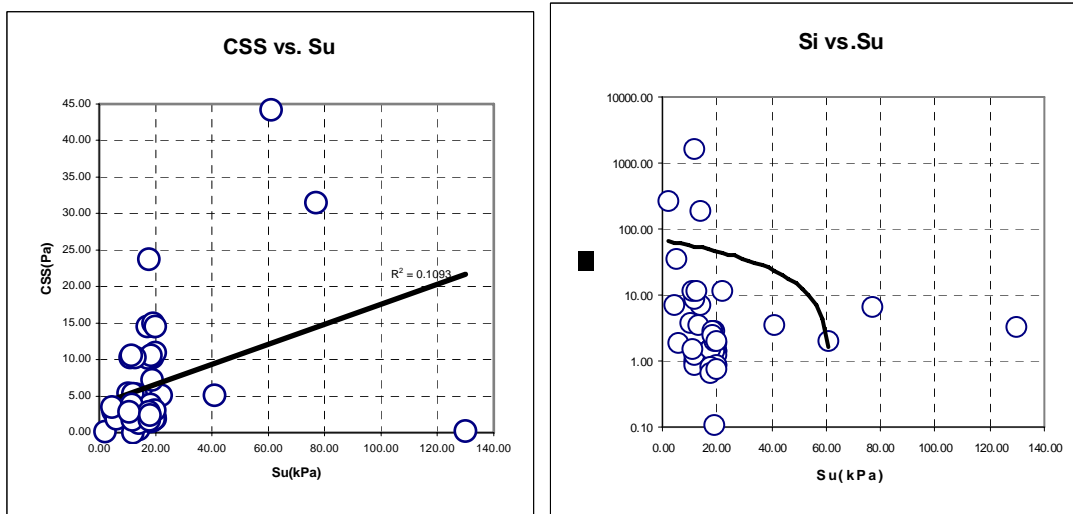


Figure 9.13: Erosion properties as a function of undrained shear strength.

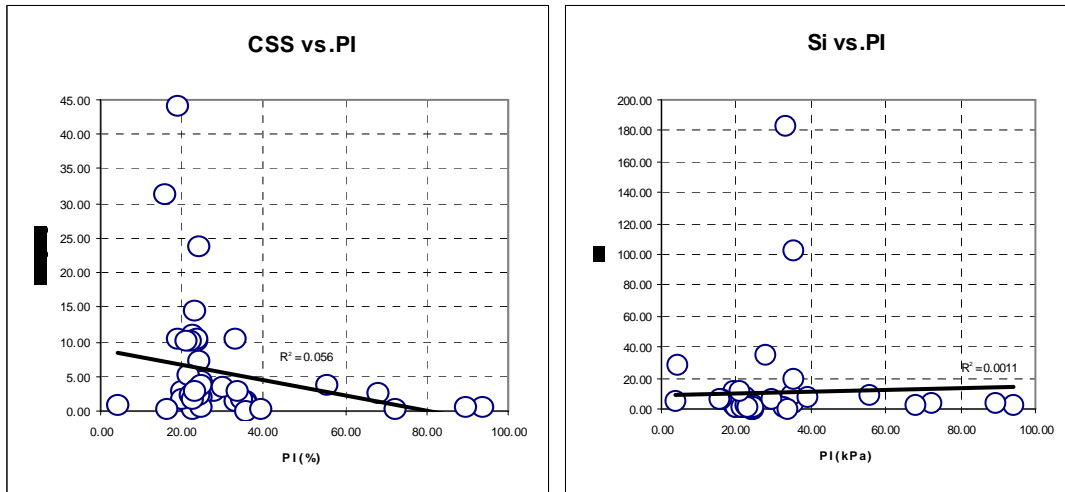


Figure 9.14: Erosion properties as a function of plasticity index.

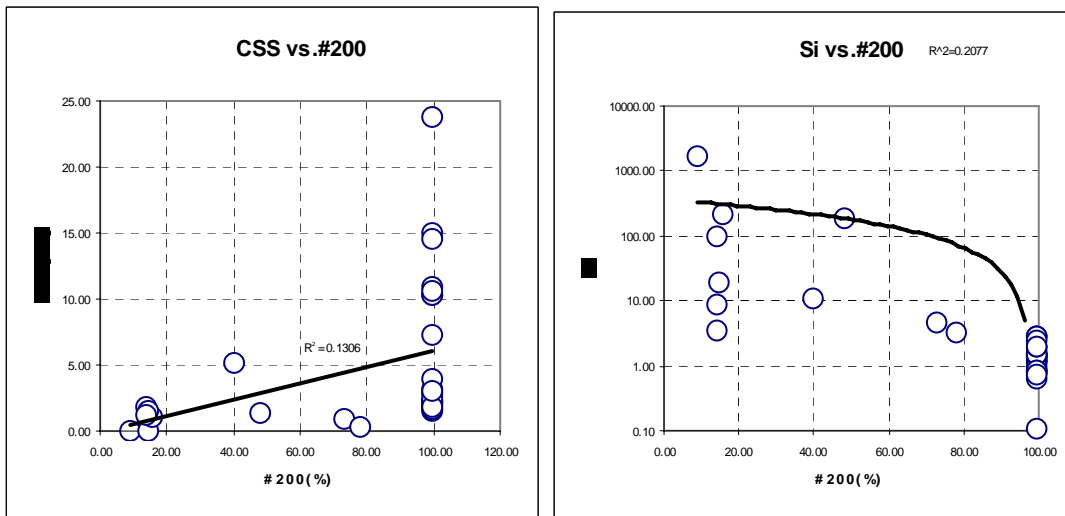
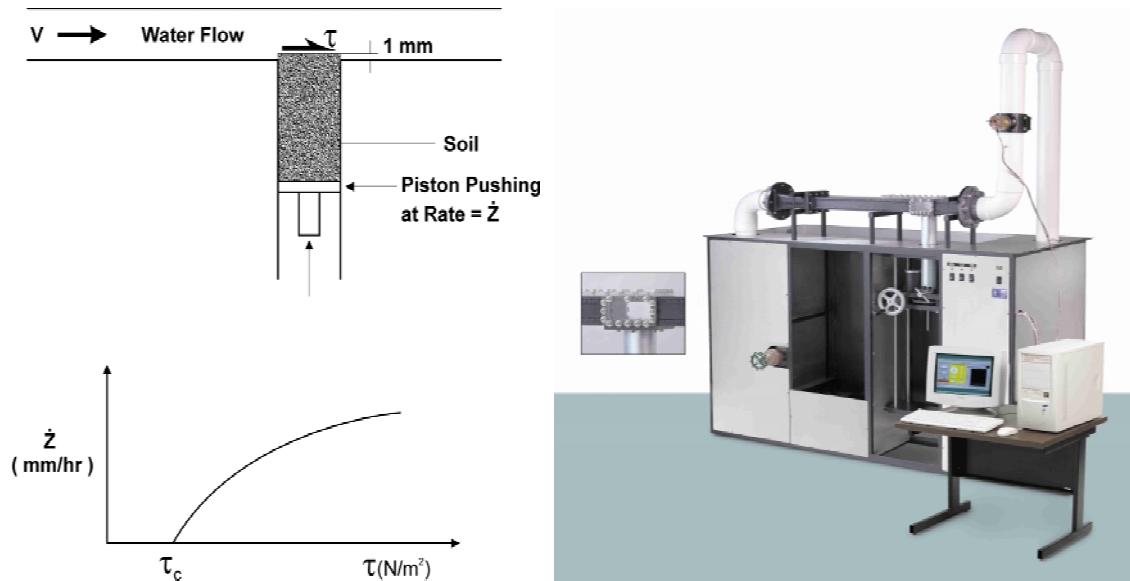
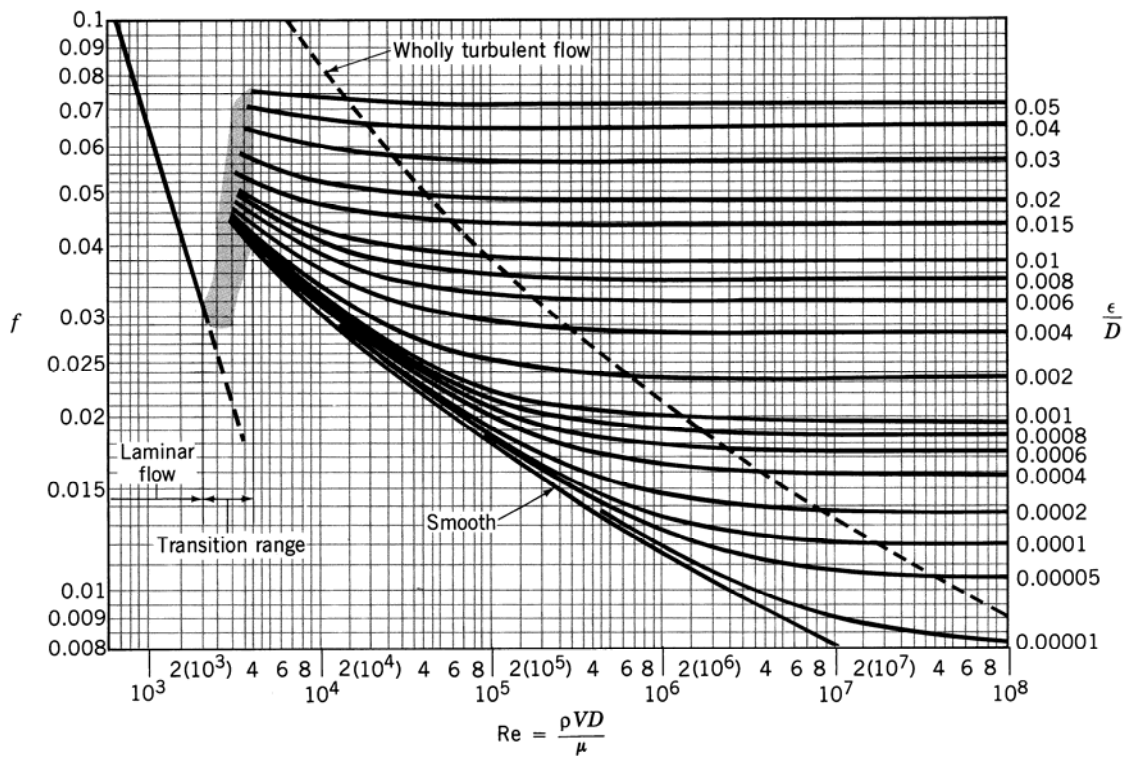


Figure 9.15: Erosion properties as a function of percent passing sieve #200.



Source: Briaud et al (2001)

Figure 9.16: EFA (Erosion Function Apparatus).

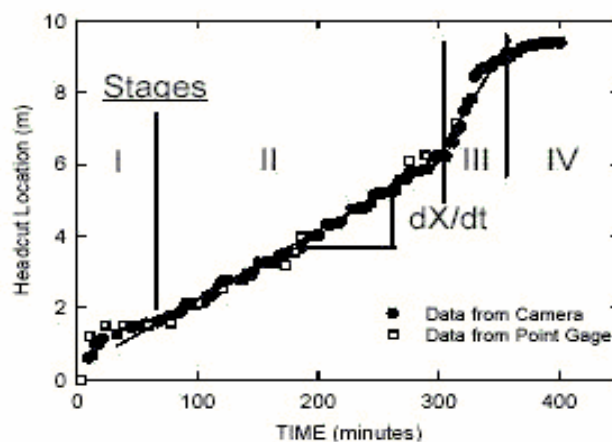


Source: Munson et al (1990)

Figure 9.17: Moody Chart.

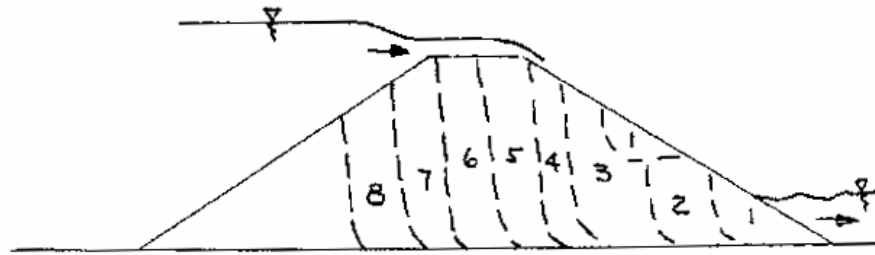
Table 9.6: Procedures for Levee Design and Construction (USACE, 2000)

Major and Minimum Requirements	
Step	Procedure
1	Conduct geological study based on a thorough review of available data including analysis of aerial photographs. Initiate preliminary subsurface explorations.
2	Analyze preliminary exploration data and from this analysis establish preliminary soil profiles, borrow locations, and embankment sections.
3	Initiate final exploration to provide: <ul style="list-style-type: none"> a. Additional information on soil profiles. b. Undisturbed strengths of foundation materials. c. More detailed information on borrow areas and other required excavations.
4	Using the information obtained in Step 3: <ul style="list-style-type: none"> a. Determine both embankment and foundation soil parameters and refine preliminary sections where needed, noting all possible problem areas. b. Compute rough quantities of suitable material and refine borrow area locations.
5	Divide the entire levee into reaches of similar foundation conditions, embankment height, and fill material and assign a typical trial section to each reach.
6	Analyze each trial section as needed for: <ul style="list-style-type: none"> a. Underseepage and through seepage. b. Slope stability. c. Settlement. d. Trafficability of the levee surface.
7	Design special treatment to preclude any problems as determined from Step 6. Determine surfacing requirements for the levee based on its expected future use.
8	Based on the results of Step 7, establish final sections for each reach.
9	Compute final quantities needed; determine final borrow area locations.
10	Design embankment slope protection.



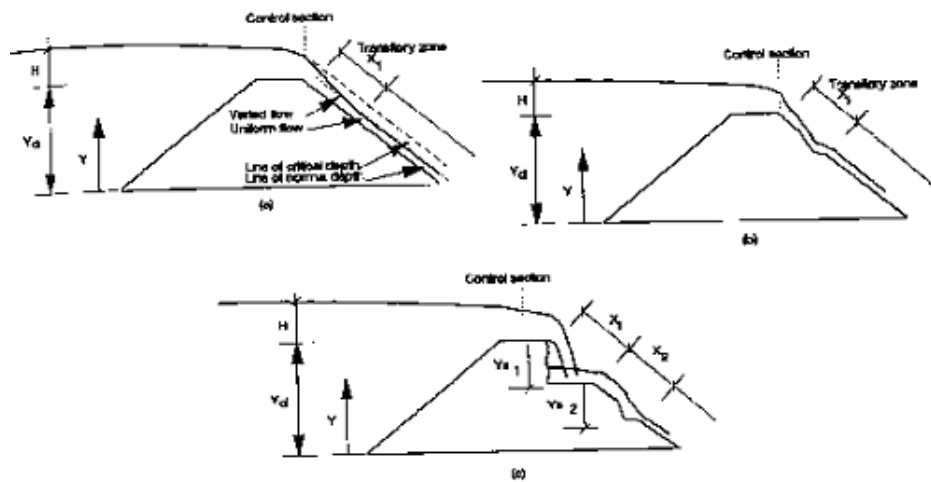
Source: Hanson et al. (2003)

Figure 9.18: Headcut Location as a Function of Time.



Source: Ralston (1987)

Figure 9.19: Stages of Progressive Erosion.



Source: AlQaser, Ruff (1993)

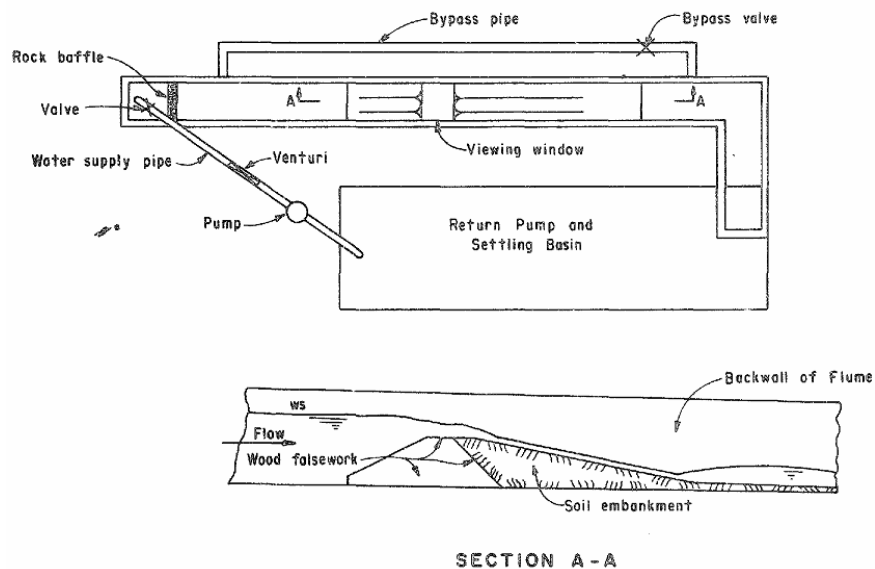
Figure 9.20: Progressive Failure of an Overtopped Embankment.

Table 9.7: Summary of Dam Break Computer Models (AlQaser, 1991)

Model (Yr of Publ)	Hydrodynamic Approach	Sediment Transport	Solution Algorithm	Breach Morphology	Characteristic Parameters	Other Features
Cristofano (1965)	Broad-crested weir hydraulic relation	Empirical relation	Manual- iterative	Constant width	Proportionality constant, angle of repose	No tailwater effects, no sloughing
Harris & Wagner (1967)		Schoklitsch bed- load formula	Numerical	Parabolic shape	Grain size, critical discharge value, breach dimensions and slope	No tailwater effects, no sloughing
BRDAM Brown & Rogers (1977, 1981)						
DAMBRK – Fread (1977)		Linear pre- determined rate of erosion		Rectangular, triangular, trapezoidal	Failure duration tim, terminal size and shape of breach	No sloughing
Lou (1981)	St. Venant system of equations	Empirical relation	Preissmann’s 4- point finite difference	Regime type relation between top width and flow rate	Coefficients of the regime relation, critical shear stress	
Ponce & Tsivoglou (1981)		Meyer-Peter and Mueller bed-load formula				
BREACH – Fread (1984)	Broad-crested weir hydraulic relation		Numerical iterative	Rectangular changing to trapezoidal	Critical shear stress grain size cohesion friction angle	Tailwater effects and sloughing are included
BEED – Singh 1989)		Einstein-Brown bed-load formula		Rectangular or trapezoidal	Friction angle, dimensionless shear stress $1/\psi$	Neglects the triggering mechanism of failure, sloughing is incorporated
Froelich (1990)		linear predetermined rate of erosion				

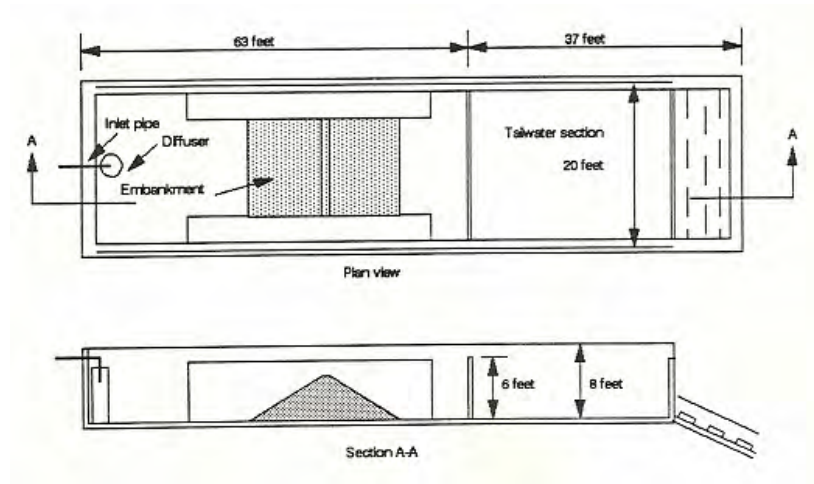
Table 9.8: Results from Flume Tests (Nairn, 1986)

Run	Description	Duration	Erosion Rate ($\text{m}^3/\text{m hr}$)
A	no bluff, composite slope, sand veneer	6 hrs.	0.0066
B	bluff, composite slope, toe sub- merged 3 cm, sand veneer	6 hrs.	0.0046
C	bluff, composite slope, toe submerged 5 cm, no sand	6 hrs.	0.0127
F	bluff, constant 1:20 slope, toe at NWL, no veneer	3.5 hrs. 6.5 hrs.	0.0109 0.0098
G	bluff, constant 1:20 slope, toe at NWL, sand veneer	1 hr.	negligible, profile armoured with sand



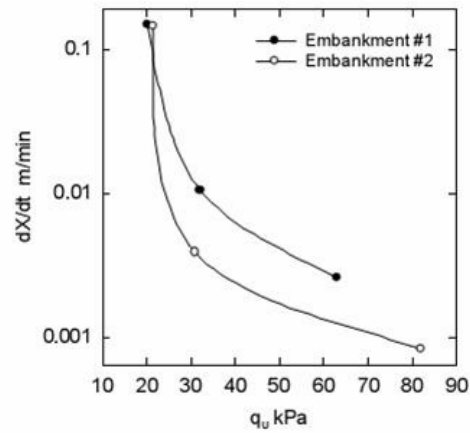
Source: Dodge (1988)

Figure 9.21: Laboratory Test Facility.



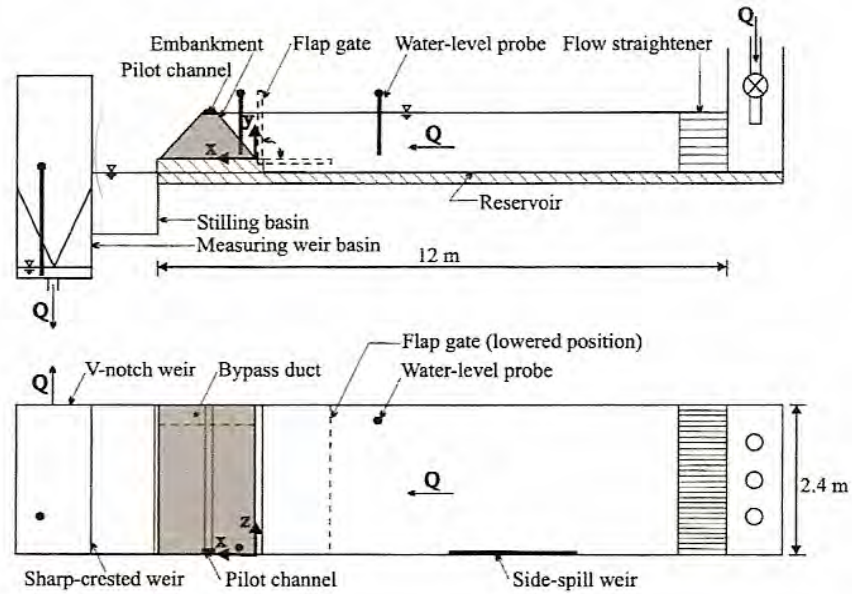
Source: AlQaser (1991)

Figure 9.22: Testing Facility.



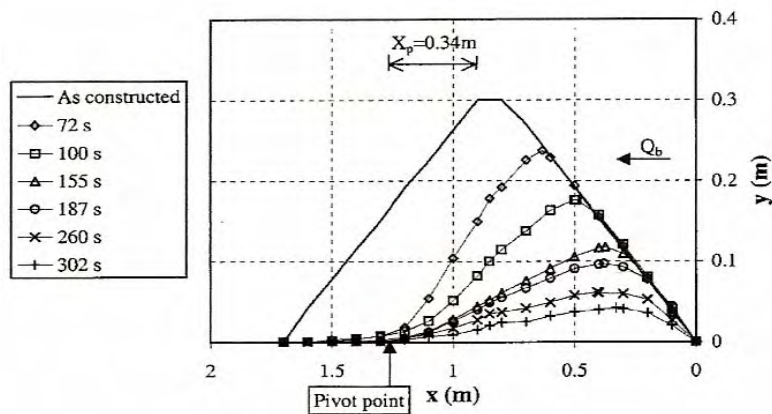
Source: Hanson et al. (2001)

Figure 9.23: Migration Rate vs. Unconfined Compression Tests.



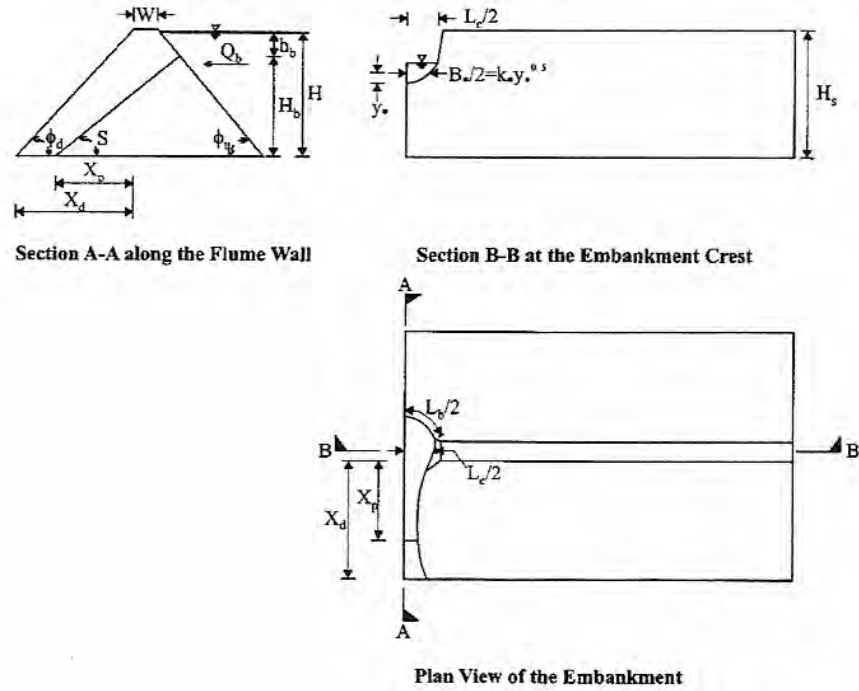
Source: Coleman et al. (2002)

Figure 9.24 : Experimental Setup.



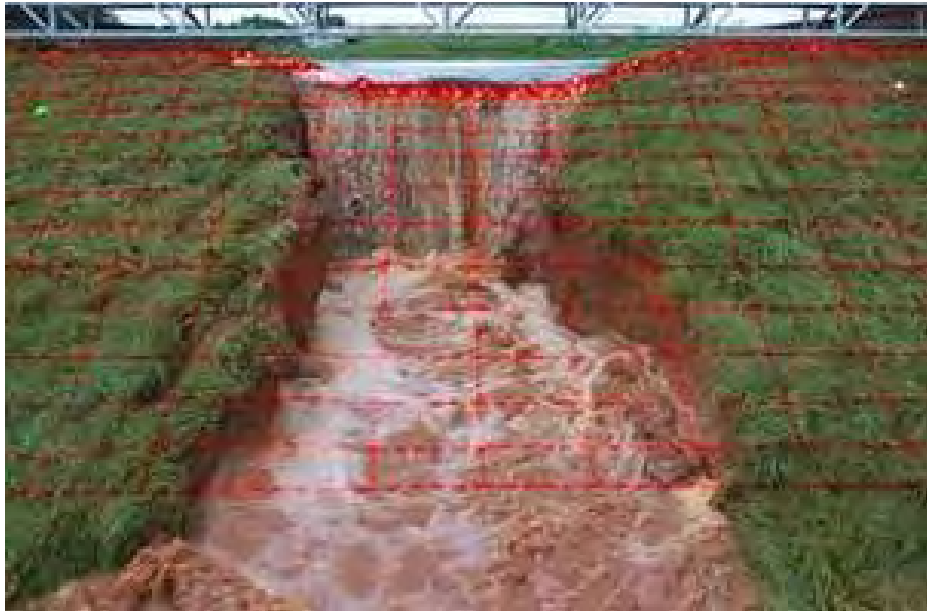
Source: Coleman et al. (2002)

Figure 9.25: Longitudinal Profiles Along Breach Channel Centerline for Medium-Sand Embankment.



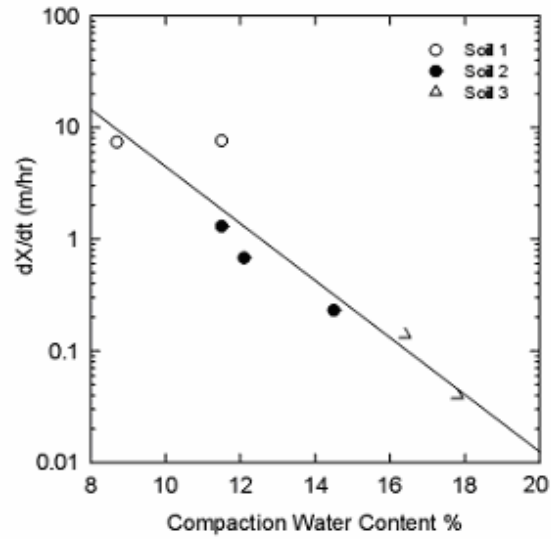
Source: Coleman et al. (2002)

Figure 9.26: Geometry Parameters for Breached Embankment.



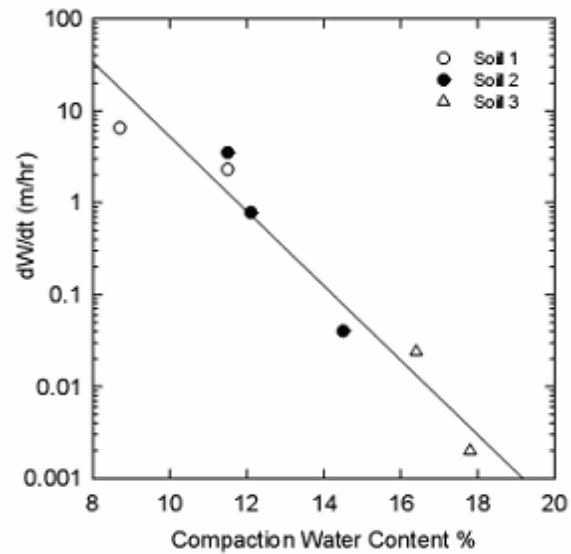
Source: Hanson et al. (2003)

Figure 9.27: Photographic Measurements of Erosion Width.



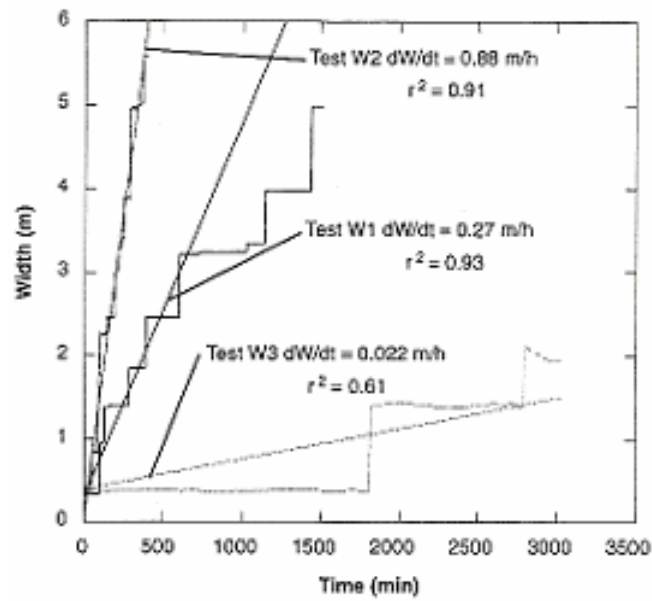
Source: Hanson et al. (2003)

Figure 9.28: Headcut Migration Rate vs. Compaction Water Content.



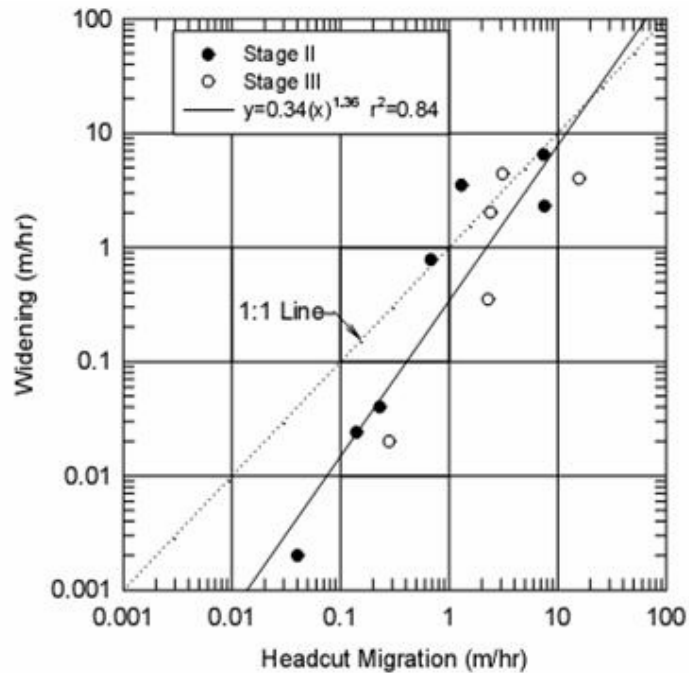
Source: Hanson et al. (2003)

Figure 9.29: Rate of Erosion Widening vs. Compaction Water Content.



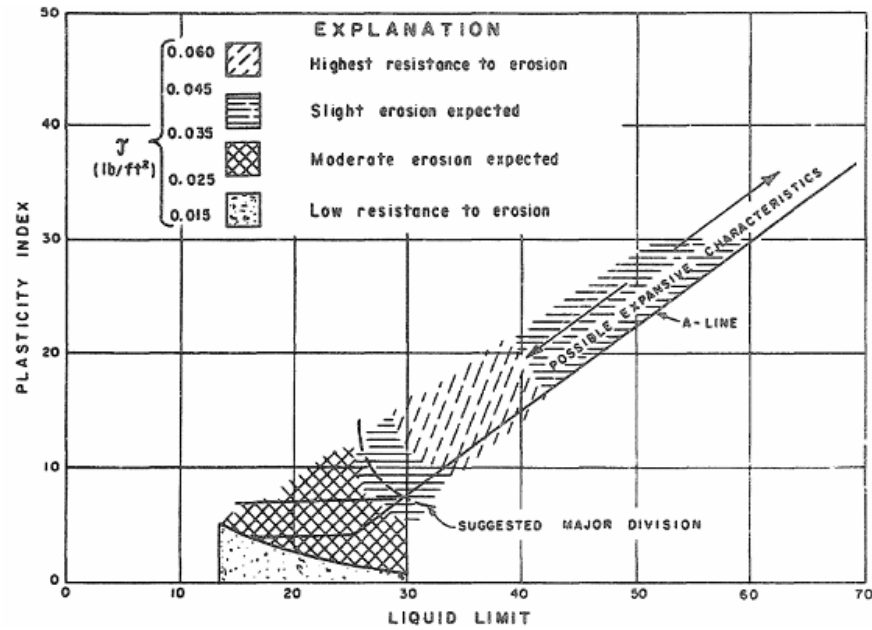
Source: Hunt et al. (2004)

Figure 9.30: Breach Width vs. Time.



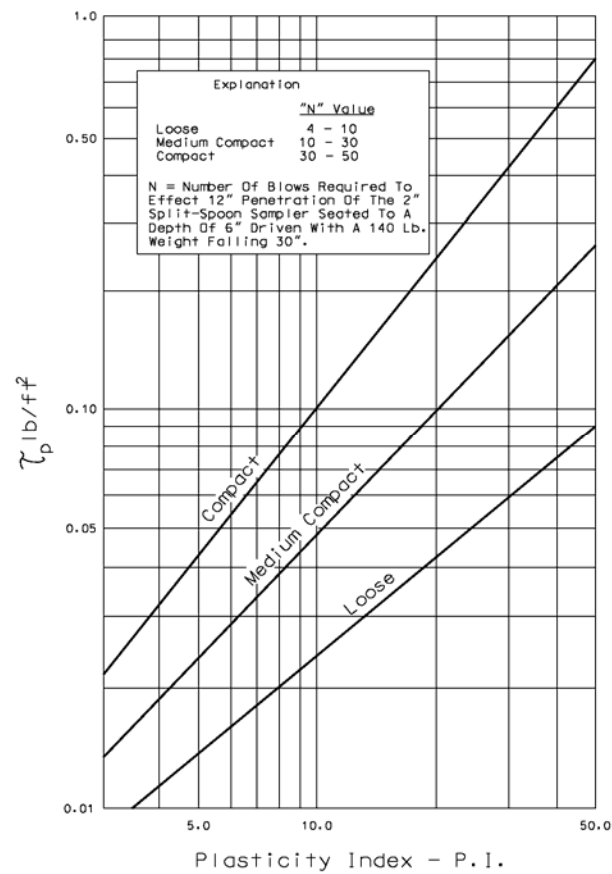
Source: Hanson et al., (2003)

Figure 9.31: Headcut Migration Rate vs. Rate of Widening.



Source: Dodge (1988)

Figure 9.32: Erosion Characteristics with respect to Plasticity.



Source: Chen & Cotton (1998)

Figure 9.33: Permissible Shear Stress for Cohesive Soils.



Figure 9.34: Difference in erosion resistance between grass cover and no grass cover.

RANGES OF SHEAR (pounds per square foot) BY DEPTH AND SLOPE

Note: This is Example A of a chart for a region with 30-40 inches of rainfall a year (good veg.)
Generally "good soil" for vegetation growth. Soil silt, sand, clay mixture.
The user should further refine this Example to a specific area of use.

Note: This assumes a "hydraulically wide" channel with generally straight alignment
shear (psf) = unit wt of water (pcf) * depth (ft) * slope (ft/ft)
If the channel is "hydraulically narrow" then replace D with R (area / wp)

Shear around bends can be greater than these values depending upon ratio of bend radius to bottom width.
This chart is intended to be used as a quick visual guide and does not take the place of individual site analysis.
The divisions between the different levels are "fuzzy" and should not be taken literally off of this chart.

Legend: Tan = soil can handle under 0.5 psf (depending upon type of soil)
Green = grasses under 3 or 4 psf (depending upon type and condition of vegetation)
Yellow = soft armor, geosynthetic matting, under 6 to 8 psf
Red = hard armor, over 7 to 8 psf

Note: The colored "ranges" change from Example A to B to C

D (ft)	0.001	0.002	0.003	0.004	0.005	0.006	0.007	0.008	0.009	0.010	0.012	0.014	0.016	0.018	0.020	0.025	0.030	0.035	0.040	0.045	0.050	0.055	0.060
0.1	0.01	0.01	0.02	0.02	0.03	0.04	0.04	0.05	0.06	0.06	0.07	0.09	0.10	0.11	0.12	0.16	0.19	0.22	0.25	0.28	0.31	0.34	0.37
0.3	0.02	0.04	0.06	0.07	0.09	0.11	0.13	0.15	0.17	0.19	0.22	0.26	0.30	0.34	0.37	0.47	0.56	0.66	0.75	0.84	0.94	1.03	1.12
0.5	0.03	0.06	0.09	0.12	0.16	0.19	0.22	0.25	0.28	0.31	0.37	0.44	0.50	0.56	0.62	0.78	0.84	1.09	1.25	1.40	1.55	1.72	1.87
0.7	0.04	0.09	0.13	0.17	0.22	0.26	0.31	0.35	0.39	0.44	0.52	0.61	0.70	0.79	0.87	1.08	1.31	1.53	1.75	1.97	2.18	2.40	2.62
1.0	0.06	0.12	0.19	0.25	0.31	0.37	0.44	0.50	0.56	0.62	0.75	0.87	1.00	1.12	1.25	1.56	1.87	2.18	2.50	2.81	3.12	3.43	3.74
1.2	0.07	0.15	0.22	0.30	0.37	0.45	0.52	0.60	0.67	0.75	0.90	1.05	1.20	1.35	1.50	1.87	2.25	2.62	3.00	3.37	3.74	4.12	4.49
1.4	0.09	0.17	0.26	0.35	0.44	0.52	0.61	0.70	0.78	0.87	1.05	1.22	1.40	1.57	1.78	2.28	2.68	3.08	3.48	3.88	4.28	4.68	5.08
1.6	0.10	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90	1.00	1.20	1.40	1.60	1.80	2.00	2.50	3.00	3.49	3.99	4.49	4.99	5.49	5.99
1.8	0.11	0.22	0.34	0.45	0.56	0.67	0.79	0.90	1.01	1.12	1.35	1.57	1.80	2.02	2.25	2.81	3.37	3.83	4.29	4.75	5.21	5.67	6.13
2.0	0.12	0.25	0.37	0.50	0.62	0.75	0.87	1.00	1.12	1.25	1.50	1.75	2.00	2.25	2.50	3.12	3.74	4.37	4.99	5.62	6.24	6.86	7.49
2.2	0.14	0.27	0.41	0.55	0.69	0.82	0.96	1.10	1.24	1.37	1.65	1.92	2.20	2.47	2.75	3.43	4.12	4.80	5.49	6.18	6.86	7.55	8.24
2.4	0.15	0.30	0.45	0.60	0.75	0.90	1.05	1.20	1.35	1.50	1.80	2.10	2.40	2.70	3.00	3.74	4.45	5.24	5.99	6.74	7.48	8.24	8.99
2.6	0.16	0.32	0.49	0.65	0.81	0.97	1.14	1.30	1.46	1.62	1.95	2.27	2.60	2.92	3.24	4.06	4.87	5.63	6.49	7.30	8.11	8.92	9.73
2.8	0.17	0.35	0.52	0.70	0.87	1.05	1.22	1.40	1.57	1.75	2.10	2.45	2.80	3.14	3.48	4.37	5.24	6.12	6.99	7.88	8.77	9.61	10.46
3.0	0.19	0.37	0.56	0.75	0.94	1.12	1.31	1.50	1.68	1.87	2.25	2.62	3.00	3.37	3.74	4.68	5.62	6.55	7.49	8.42	9.35	10.30	11.25
3.2	0.20	0.40	0.60	0.80	1.00	1.20	1.40	1.60	1.80	2.00	2.40	2.80	3.18	3.56	3.94	4.99	5.99	6.99	7.98	8.98	9.98	10.98	11.98
3.4	0.21	0.42	0.64	0.85	1.06	1.27	1.49	1.70	1.91	2.12	2.55	2.97	3.38	3.82	4.24	5.30	6.36	7.43	8.49	9.55	10.61	11.67	12.73
3.6	0.22	0.45	0.67	0.90	1.12	1.35	1.57	1.80	2.02	2.25	2.70	3.14	3.59	4.04	4.49	5.62	6.74	7.86	8.98	10.11	11.23	12.36	13.48
3.8	0.24	0.47	0.71	0.95	1.19	1.42	1.66	1.90	2.13	2.37	2.85	3.32	3.79	4.27	4.74	5.93	7.11	8.30	9.48	10.67	11.85	13.04	14.23
4.0	0.25	0.50	0.75	1.00	1.25	1.50	1.75	2.00	2.25	2.50	3.00	3.49	3.99	4.49	4.99	6.24	7.49	8.74	9.98	11.23	12.48	13.73	14.98
4.2	0.26	0.52	0.79	1.05	1.31	1.57	1.83	2.10	2.36	2.62	3.14	3.67	4.19	4.72	5.24	6.55	7.86	9.17	10.48	11.78	13.10	14.41	15.72
4.4	0.27	0.55	0.82	1.10	1.37	1.65	1.92	2.20	2.47	2.75	3.29	3.84	4.39	4.94	5.49	6.86	8.24	9.61	10.98	12.36	13.73	15.10	16.47
4.6	0.29	0.57	0.86	1.15	1.44	1.72	2.01	2.30	2.58	2.87	3.44	4.02	4.59	5.17	5.74	7.16	8.51	9.85	11.18	12.52	13.85	15.19	16.52
4.8	0.30	0.60	0.90	1.20	1.50	1.80	2.10	2.40	2.70	3.00	3.59	4.19	4.79	5.39	5.99	7.49	8.98	10.48	11.98	13.48	14.98	16.47	17.97
5.0	0.31	0.62	0.94	1.25	1.56	1.87	2.18	2.50	2.81	3.12	3.74	4.37	4.99	5.62	6.24	7.80	9.35	10.92	12.48	14.04	15.50	17.15	18.72
5.5	0.34	0.69	1.03	1.37	1.72	2.06	2.40	2.75	3.09	3.43	4.12	4.80	5.49	6.18	6.86	8.55	10.30	12.01	13.73	15.44	17.15	18.98	20.68
6.0	0.37	0.75	1.12	1.50	1.87	2.25	2.62	3.00	3.37	3.74	4.49	5.24	5.99	6.74	7.49	9.36	11.23	13.10	14.98	16.85	18.72	20.59	22.46
6.5	0.41	0.81	1.22	1.62	2.03	2.43	2.84	3.24	3.65	4.06	4.87	5.68	6.49	7.30	8.11	10.14	12.17	14.20	16.22	18.25	20.28	22.31	24.34
7.0	0.44	0.87	1.31	1.75	2.19	2.62	3.06	3.49	3.93	4.37	5.24	6.12	6.99	7.86	8.74	10.92	13.10	15.29	17.47	19.66	21.84	24.02	26.21

shear.xls Shear A

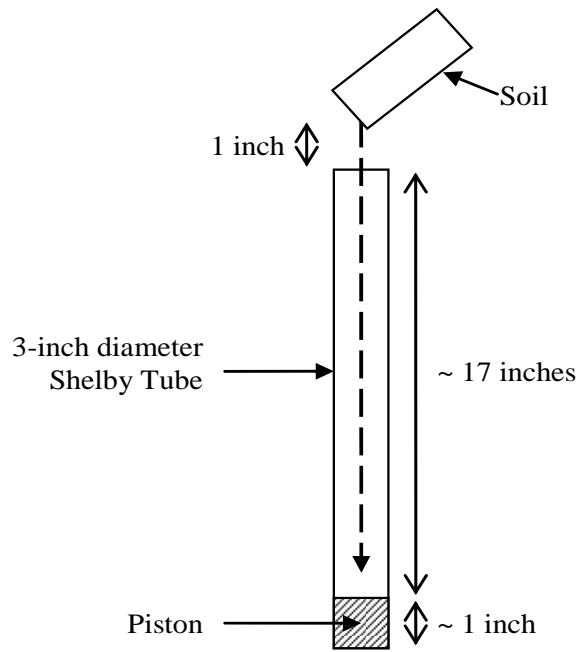
R Chan Associates - Austin

Figure 9.35: Range of shear stresses allowable on slopes for different covers.

Source: Johnston (2003)



	pH	Salinity (ppm)
Sample S11	8.61	3287
Sample S15	8.09	4199
Typical sea water	7.9	30000 to 35000
Typical tap water	7.0	500



Sample Preparation
(Note: Dimensions Indicative Only)

Figure 9.37: Soil preparation by re-compaction for bag samples.

EFA Test Results for Sample No. S4-(0-0.5ft)-LC-SW

Sample Type: Bulk Sample

Water Salinity: 36.1 PPT (Salt Water)

Compaction Effort: Low = 1.6% Modified Proctor Compaction

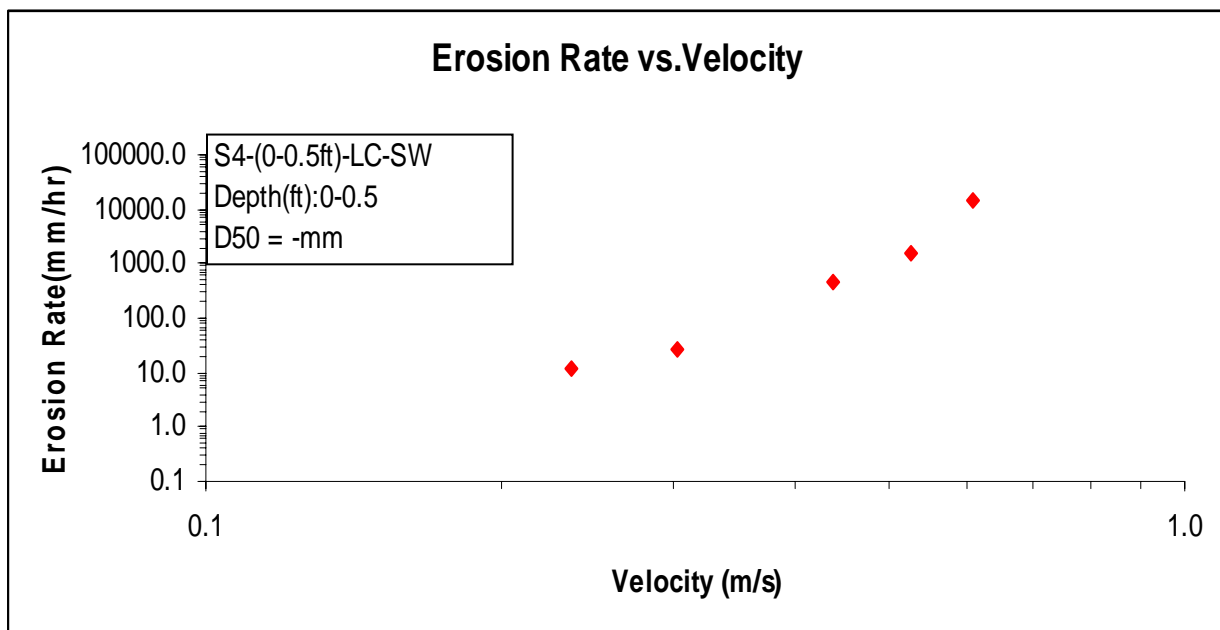
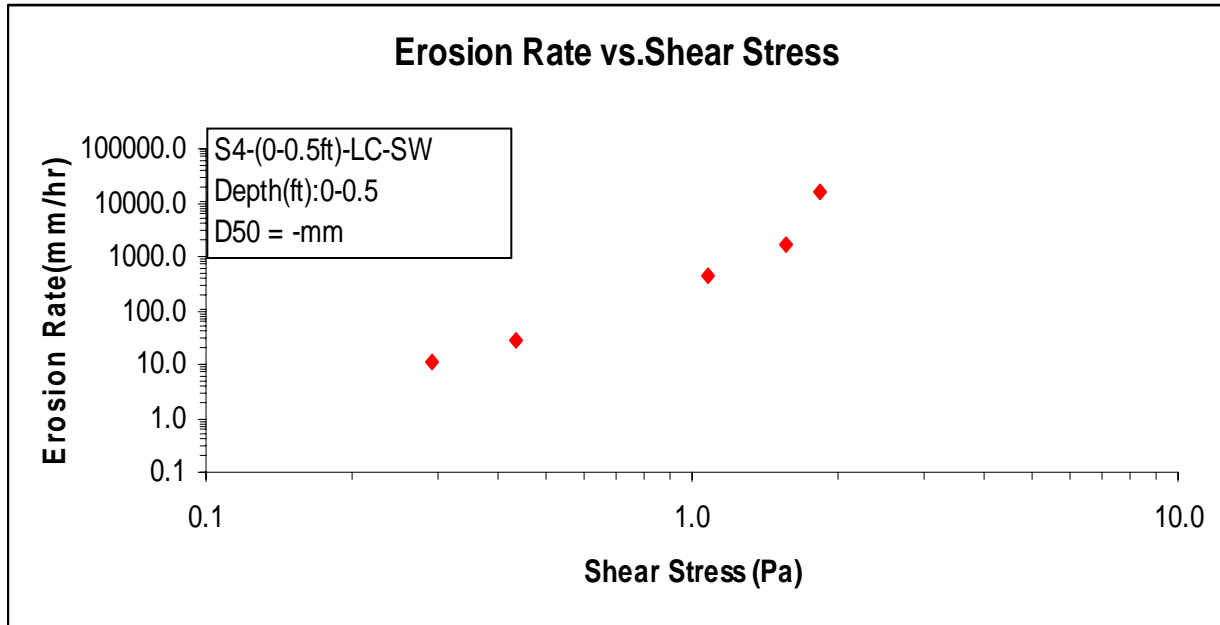


Figure 9.38: EFA test results for sample S4 (0-0.5 ft), low compaction, salt water.

EFA Test Results for Sample No. S3-B3-(0-1ft)-SW

Sample Type: Shelby Tube
Water Salinity: 36.4 PPT (Salt Water)
Compaction Effort: N/A

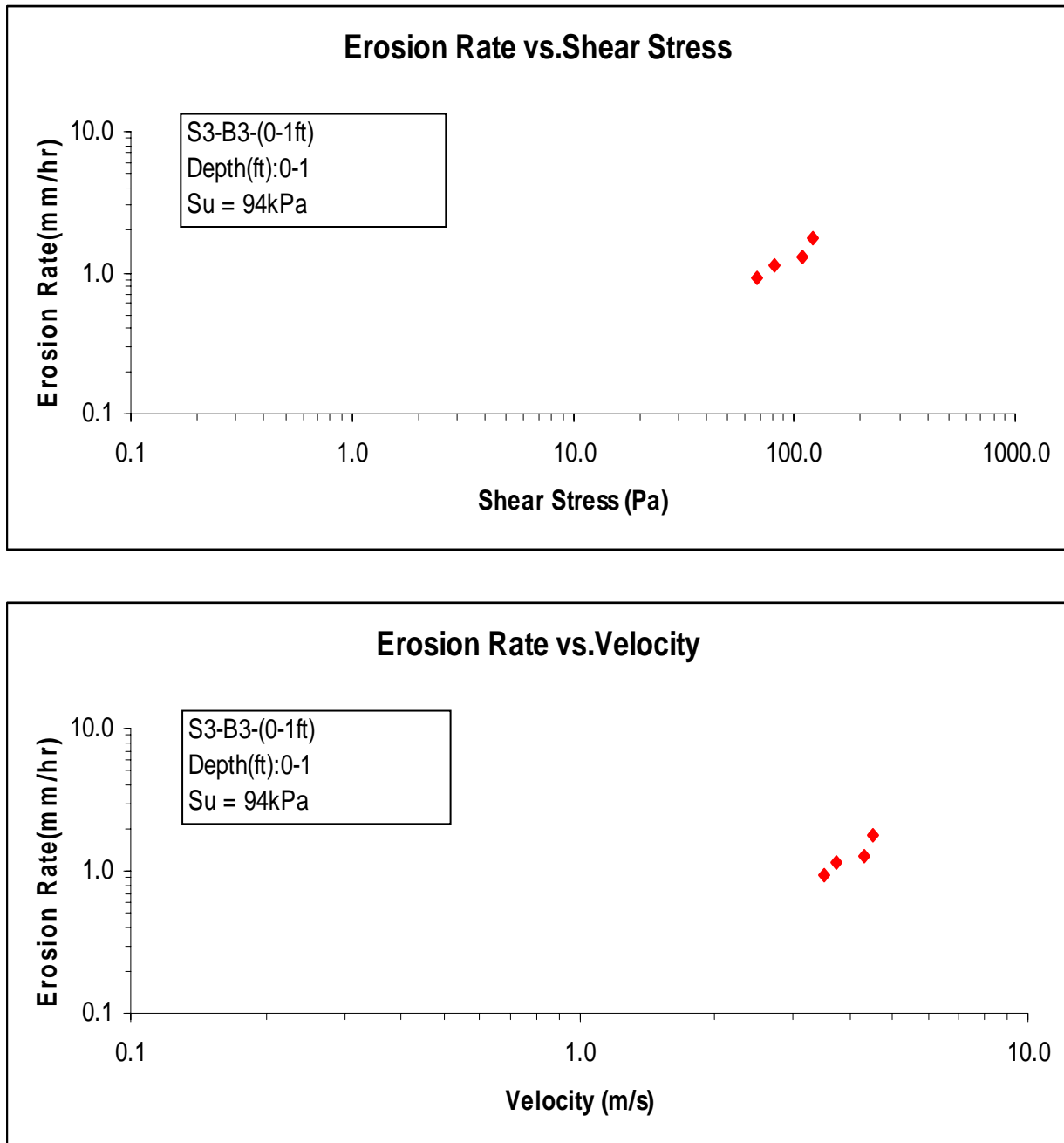


Figure 9.39: EFA test results for sample S3-B3 (0-1 ft), salt water.

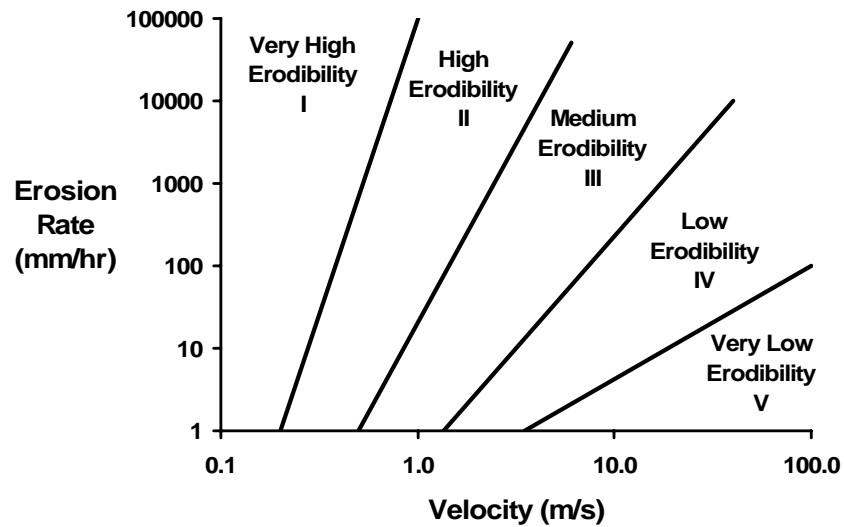


Figure 9.40: Erosion Chart.

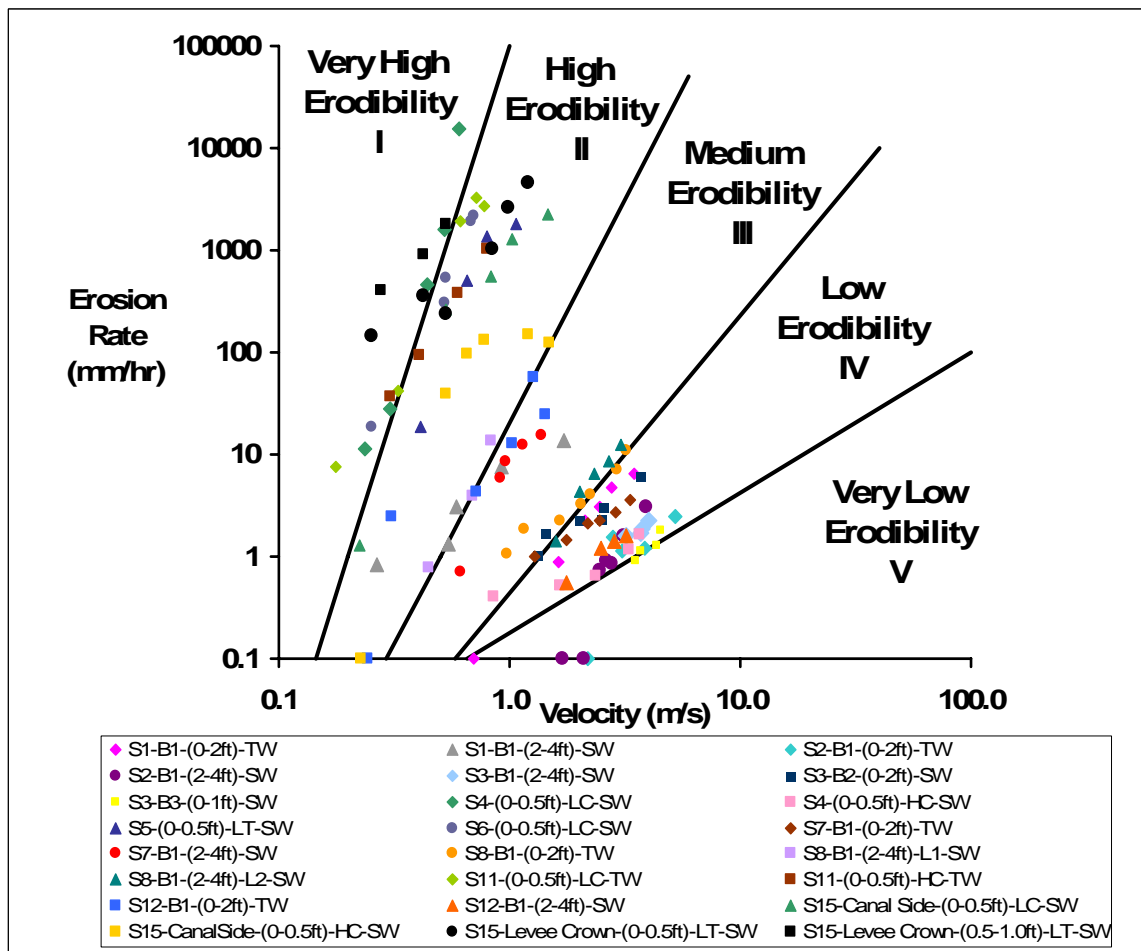


Figure 9.41: EFA test results for 24 levee samples.

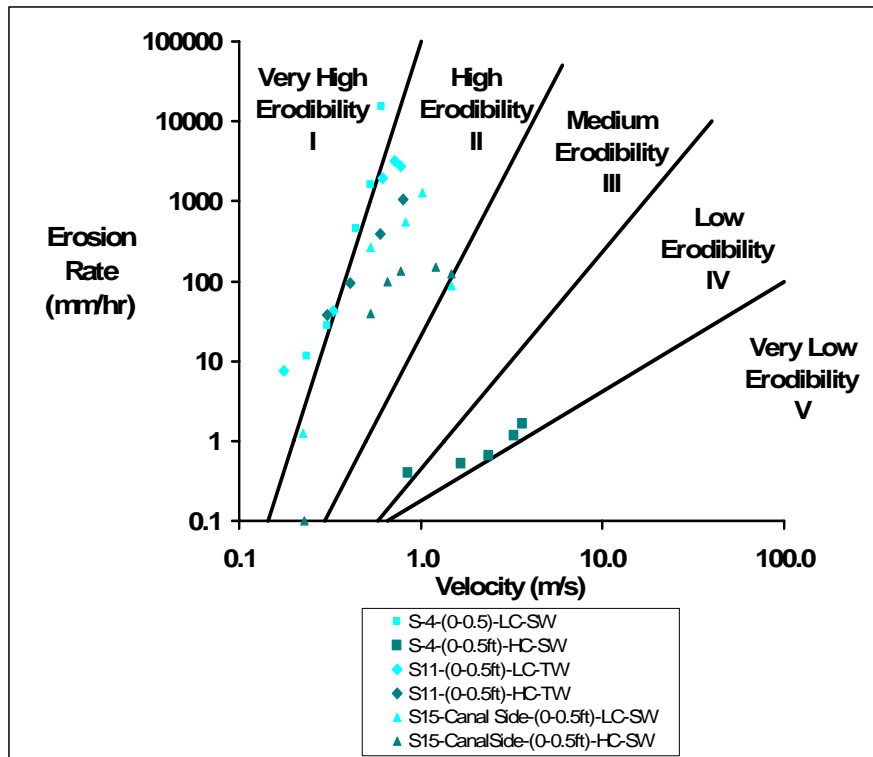


Figure 9.42: Influence of compaction on erodibility.

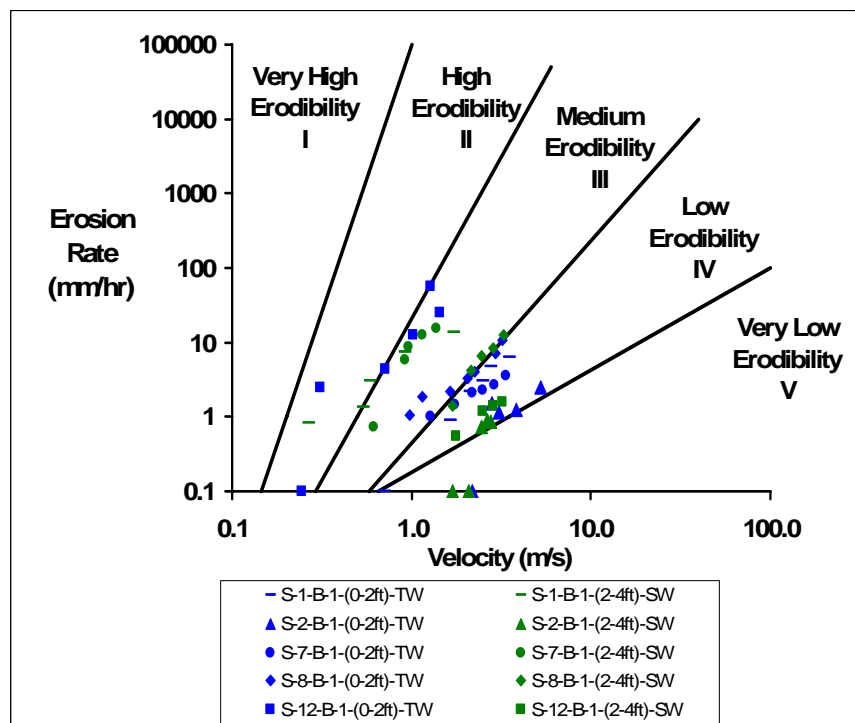


Figure 9.43: Influence of water salinity on erodibility.

Table 9.10: Results of the index property tests.

Sample	Soil Description	Classification	γ_t (kN/m3)	γ_{dry} (kN/m3)	w (%)	Tested at Texas A&M University					Tests Performed by STE			
						% fines	LL	PL	PI	Organic Content%	% fines	LL	PL	PI
S1-B1-(0-2ft)-TW	Clay with hard clay grain mixture	CH	20.23	15.37	31.66	-	71	25	46	3.09	89.9	65	22	43
S1-B1-(2-4ft)-SW	Clay with rootlets	CH	19.10	15.69	21.77	-	56	19	37	1.91				
S2-B1-(0-2ft)-TW	Clay with rootlets	CL	19.74	17.00	16.11	-	46	17	29	16.94	67.2	49	17	32
S2-B1-(2-4ft)-SW	Clay	CL	20.26	16.71	21.23	69.1	41	16	25	1.62				
S3-B1-(2-4ft)-SW	Clay	CL-CH	17.60	13.86	27.00	-	48	17	31	2.50				
S3-B2-(0-2ft)-SW	Clay with some sand	CH	20.20	17.26	31.66	-	69	23	46	-	90.3	54	19	35
S3-B3-(0-1ft)-SW	Clay	CL-CH	17.16	13.95	23.00	-	32	12	20	2.60				
S4-(0-0.5ft)-LC-SW	Clay with some sand	CL	13.87	10.42	33.14	90.5	60	30	30	8.16	-	-	-	-
S4-(0-0.5ft)-HC-SW	Clay with some sand		17.69	13.23	33.14									
S5-(0-0.5ft)-LT-SW	Silt-Clay		21.85	18.15	20.40	54.4	-	-	-	0.69	-	-	-	-
S6-(0-0.5ft)-LC-SW	Sand w/Some Clay	SP	13.45	12.79	5.21	8.9	-	-	-	0.71				
S7-B1-(0-2ft)-TW	Clay	CH	17.39	13.73	26.65	-	68	24	44	3.78	90.1	78	32	46
S7-B1-(2-4ft)-SW	Clay with hard clay grain mixture	CH	16.52	13.42	23.04	-	-	-	-	7.14				
S8-B1-(0-2ft)-TW	Clay with 1.5" thick grass on top of sample	CH	17.71	13.38	32.34	-	-	-	-	-				
S8-B1-(2-4ft)-L1-SW	Clay with 2 layers	CH	18.74	14.00	33.87	-	-	-	-	2.28	97.3	85	36	49
S8-B1-(2-4ft)-L2-SW	Clay with 2 layers	CH	18.74	14.00	33.87	-	54	21	33	15.37				
S11-(0-0.5ft)-LC-TW	Sand	SP	12.30	12.23	1.02	0.1	-	-	-	0.32	-	-	-	-
S11-(0-0.5ft)-HC-TW	Sand	SP	13.26	13.12	1.02		-	-	-	0.35				
S12-B1-(0-2ft)-TW	Clay with decomposed wood	CH	14.77	10.19	44.94	-	67	27	40	16.91	92	67	21	46
S12-B1-(2-4ft)-SW	Clay	MH-CH	17.56	12.64	38.94	-	58	32	26	5.28				
S15-CanalSide-(0-0.5ft)-LC-SW	Sand w/Some Clay	SM	13.85	12.21	13.43	29.9	-	-	-	1.28				
S15-CanalSide-(0-0.5ft)-HC-SW	Sand w/some clay		19.63	17.31	13.43		-	-	-					
S15-LeveeCrown-(0-0.5ft)-LT-SW	Sand w/Some Clay		13.29	11.94	11.29		-	-	-	2.16				
S15-LeveeCrown-(0.5-1ft)-LT-SW	Sand w/Some Clay		13.57	12.46	8.93		-	-	-	1.01				

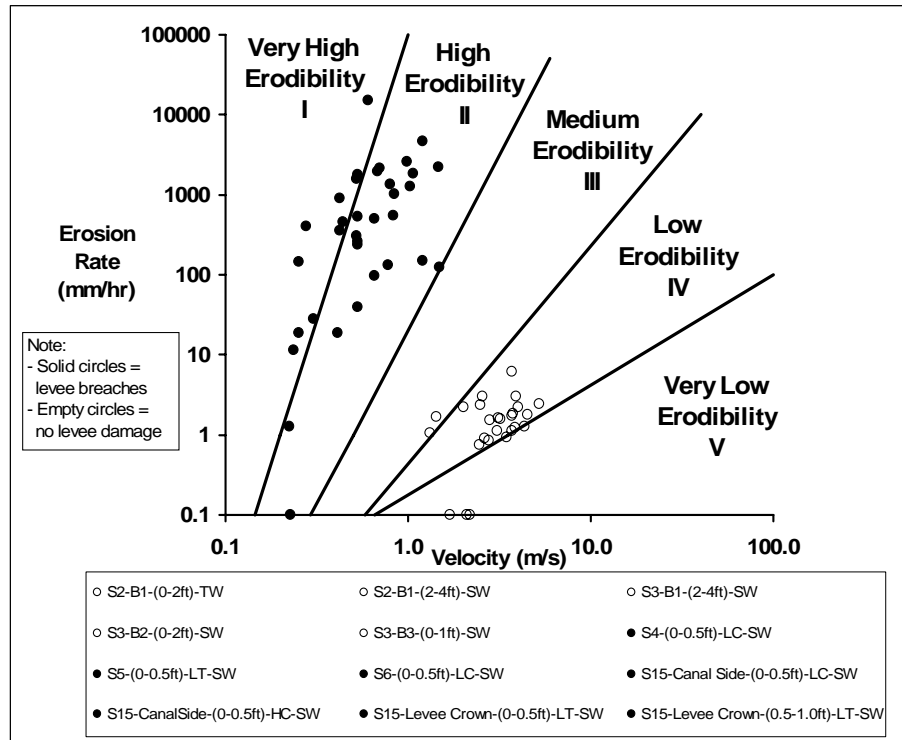


Figure 9.44: EFA test results and overtopping levee failure/no failure chart.

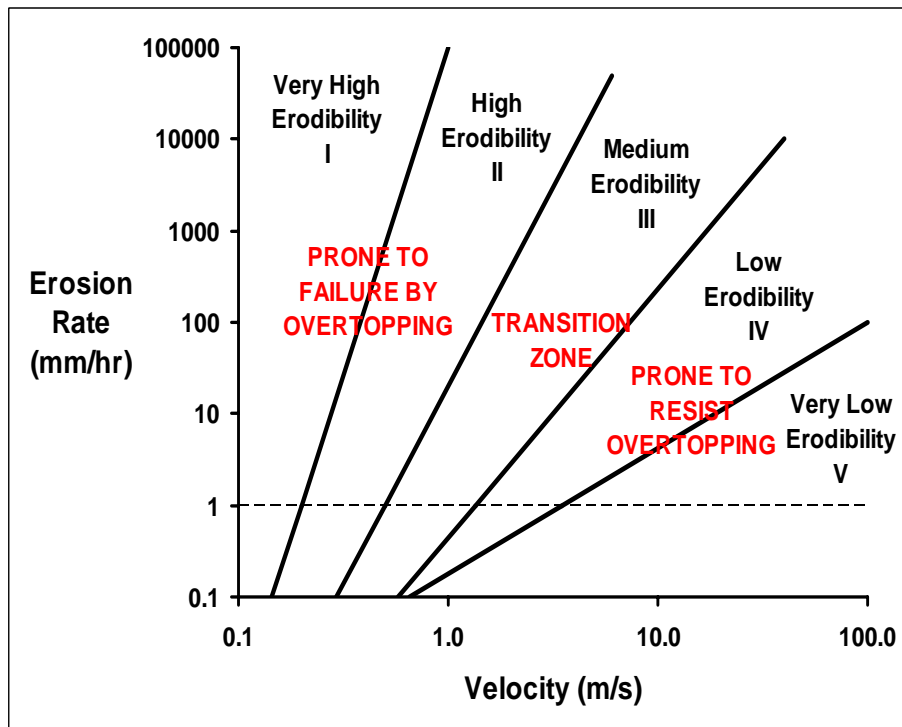


Figure 9.45: Proposed guidelines for levee overtopping.

CHAPTER TEN: ENGINEERING OVERVIEW; EARTHEN LEVEES AND FLOODWALLS

10.1. Overview

The vast majority of flood protection for the greater New Orleans area is strongly dependent upon the presence and ability of earthen levees to separate large water bodies, such as Lake Pontchartrain, Lake Borgne, the Mississippi River, and the Gulf of Mexico, and appurtenant channels and canals, from inundating developed land areas and causing flooding of homes and businesses. Earthen levee flood protection systems not having redundancy can be viewed as series systems, where failure at one location, or failure of one component, can result in catastrophic failure of the entire flood protection system and result in tragic loss of life, damage to fundamental infrastructure (basic services such as water, sewage, and electricity), and substantial devastation and economic impact to the immediate and surrounding regions. These systems can be in place for a short duration (a few years) or for a very long duration (hundreds of years). In order to ensure the desired level of flood protection system performance, identification and mitigation of “weak links” in the system is crucial in order to maintain long-term system integrity.

The earthen levees are supplemented and extended at many locations by means of more “structural” components comprised of concrete and steel. Steel sheetpile curtains are routinely used either to extend a “cut off” barrier to retard underseepage flow beneath levees, or to provide support for reinforced concrete floodwalls at the crests of earthen levees. In some cases, the sheetpile curtains are extended vertically above the earthen crests without concrete to simply extend the crest elevation as an interim measure until a more permanent crest raising can be implemented. The concrete floodwalls are used to achieve increased crest height without the extra weight of additional earthen levee fill, and/or without the need to widen the earthen levee embankment section to accommodate additional earthen levee fill in situations where the available “footprint” is limited. Concrete walls are also employed to provide frames for gates (usually steel gates) that can be opened to allow traffic to pass through (e.g.: automobiles, trains, ships, etc.) and then closed when storms arrive.

Few studies have systematically analyzed actual long-term performance of earthen levees and/or composite levee-floodwall systems to confirm effective design parameters, assumed loading conditions, and actual performance after major flooding events. Additionally, evaluations of component transitions (i.e. earthen levee to concrete structure transitions), erodeability overtopping, wave-scour, and effective inspection programs have not been well documented and are critical components for high reliability flood protection systems.

Chapter 10 builds upon the technical lessons from the previous chapters, and establishes additional findings as well as background to facilitate the presentation of “lessons learned” and “recommendations” with regard to design and construction of these types of regional flood protection systems that will then be presented and discussed in Chapter 11.

The main goals of this Chapter are to: (1) provide a brief overview of some of the principal design procedures and standards employed in the development of pre-Katrina regional

flood protection system, (2) identify several critical “weak link” features not adequately addressed in current design methods widely used in that region so that appropriate design modifications can be implemented to improve levee performance, (3) establish an erodeability testing methodology which can be used to assess existing earthen levees, (4) present some limited comments regarding “unwatering” (pumping), and (5) present comments and observations regarding emergency and interim levee and floodwall reconstruction efforts in the wake of hurricane Katrina.

10.2. Potential Levee Failure Mechanisms

There are numerous failure mechanisms that can result in the failure and breaching of earthen levees and/or floodwalls, and the resultant catastrophic flooding of protected areas. These failure mechanisms can occur as a single mode, or as a combination several different types of failure modes acting in unison. Levees can fail as a result of damage to the levee itself, if the foundation on which the levee is constructed fails, or as a result of failure of a floodwall for a composite levee/floodwall section. An abbreviated overview of many of the potential failure mechanisms of interest in the greater New Orleans area is presented here:

10.2.1. Structural Causes

This category includes potential failure mechanisms where the dominant issue is either the strength and stability of the levee embankment and/or foundation soils, or the structural capability of “structural” elements (e.g. sheetpile curtains, floodwalls, or gates) and/or their interaction with the levees and foundations soils that support them. Such mechanisms include:

Slope Instability – If the levee embankment soils and/or the underlying foundation materials that support the levee are weak, or become destabilized, a slope failure can develop and result in catastrophic failure of the levee. Slope failures can be minor or they can be significant enough to result in the catastrophic failure of the levee system. A number of catastrophic failures occurred during hurricane Katrina due to this mechanism. Slope failures can be subdivided into separable classes as

- (a) Bearing capacity failure; Failure of the weak foundation soils to vertically support the weight of the levee embankment. This is most common during construction (before the foundation soils have time to consolidate and gain strength under the embankment load.) A failure of this type just recently occurred on a section of levee under reconstruction in Plaquemines Parish on May 29, 2006.
- (b) Lateral translational stability failure; Failure by sliding laterally, usually as a result of being “pushed” by elevated water pressures on the water (canal) side.
- (c) Deeper, rotational-type stability failure; This can also be caused by the “push” of elevated water on the outboard (canal) side, but these types of failures can also occur due to undercutting of the canal side of the levee by dredging operations, or by storm surge scour or river flow.

Structural Failures of Walls, Sheetpiles or Gates – Simple structural failure as a result of structural elements inability to safely bear the forces and loads exerted against them. There appear to have been no structural failures of this type, except in cases where other types of

embankment or foundation failure, or overtopping erosion and resultant lateral unbracing of floodwalls, occurred.

Structural Impacts – Structural impacts occur when physical objects collide with the levee. This can occur during storm events when boats or barges become loose from their moorings and are driven into the levee by wind or water forces, or simply from accidental boat impacts due to operator error.

10.2.2. Causes due to Hydraulic Forces

This category includes failure mechanisms where the dominant parameters involve groundwater flow and pore pressure. Among these are:

Underseepage/Instability – As shown in Figure 10.1 (red lines), if the underlying foundation materials that support the levee are adequately permeable, water can quickly travel through these porous materials as the water head differential between the outboard and inboard sides of the levee increases. This underseepage raises the pore pressures within the soils at the inboard side, and this in turn reduces the shear strengths of these soils. This can result in catastrophic failure by means of resultant slope instability (as described in the previous section).

Bottom Heave or “Blowout” - This is a variant of underseepage, but involves the hydraulic pressure, rather than simple erosion. An increase in water pressure caused by a storm surge can travel through a permeable zone in a levee’s foundation. If the water pressure exceeds the total overburden pressure at the landside toe of the levee, then the (impervious) soil overburden at that location can be displaced (heaved) by the water pressure, producing a large void into which subsequent flow will rush (rapidly exacerbating localized erosion and failure). This is often referred to as a “blowout” failure.

Erosion and Piping – As shown in Figure 10.1 (blue lines), erosion and piping occurs when the localized hydraulic gradient becomes large enough to “pull” soil grains from their location, and when there is no soil on that soil’s down-flow side that can “filter” these soil grains and thus hold them in place. Erosion and piping can be sub-divided into two sub-classes as:

(a) Exit seepage erosion and piping; This is one of the most common causes of levee failures world-wide. If underseepage flow (and/or flow laterally through the levee embankment) becomes sufficient as to raise the exit gradient, then there is little or no resistance to erosion of soil particles at the point of water exit (either low on the inboard side levee slope face, or on the ground surface at and just inboard of the levee toe). Once the seepage gradient is able to exert enough “drag” on the soil grains to overcome the stability due to their self-weight, erosion begins. As soon as erosion begins, the local flow-net rapidly converges on the “hole” that begins to develop, as the water moves towards a preferential short-cut in its effort to escape. That, in turn, increases the local gradient and thus accelerates the erosion. The result is that erosion can rapidly ‘eat back’ a tunnel (or “pipe”) beneath the levee [hence the name “piping”.] Soils from above routinely slough and fall into this rapidly developing “pipe”, but they are usually immediately washed out by the ever increasing flow until, finally, the embankment ruptures and erodes through catastrophically. An illustration of this was provided previously in Figure 8.105.

(b) Internal seepage erosion; This can occur internally within either the levee, or within the foundation soils. As water flows through these soils, smaller/finer soil particles can be “washed” out resulting in the internal erosion of the levee or foundation soil. Enough internal erosion can lead to the collapse and subsequent full-blown “wash-out” failure of the levee. For levees constructed of layers with significantly different permeabilities, the layer with the highest permeability becomes the main “conduit” by which the water flows through the levee. This concentrated flow can lead to higher water velocities through the levee and more rapid degradation. Appropriate control of soil gradation, and use of appropriate soil gradations within adjacent embankment sections (which is called “filtering”), are the keys to prevention of internal erosion.

The major design standards specifically address both internal erosion and piping as well as exit seepage erosion and piping, and levees are required to be engineered against these failure mechanisms. Mitigation strategies can include utilization of low permeability materials (such as clay), provision of underseepage “cut offs” (e.g. sheetpile curtains extending below the levees), internal drains or filters to safely “vent” pore pressures while filtering soil grains to hold them in place, widenend embankments and use of inboard-side stability berms to lengthen the flow path (and thus reduce exit gradients at the inboard side toe), control of soil gradation to prevent internal erosion, etc.

Effectively mitigating erosion and piping can be hampered by the presence of burrowing animals that can carve intricate tunnel networks within the earthen levees. Effective detection and corresponding correction of these animal-induced internal erosion channels is very challenging, and many levee failures throughout the world are a result of this failure mechanism. Exit erosion and piping can also be exacerbated by the presence of trees low on the inboard side levee faces and/or at the inboard toe. Trees that die can leave root hole paths that can exacerbate erosion and piping in this critical area. Trees that blow over during storm winds (and/or due to weakening of their roots’ foundation soils due to wetting) can suddenly leave large voids that can serve as initiation points for rapid advancement of localized exit erosion and piping. As a result, it is common practice to prevent growth of trees (as a “maintenance” issue) in this critical inboard toe area.

10.2.3. Causes Involving Surficial Erosion

These include the various forms of surface erosion which can occur due to surface water flowing over (across) or against the exposed surface of the levee.

(a) Overtopping – As shown in Figure 10.2, overtopping occurs when the water level on the outboard side of the levee exceeds the crest elevation of the levee. The inboard side of the levee acts as a spillway for the overtopping water and damage is inflicted on the levee as a result of water scour. Levees are not generally designed for overtopping and as a result, if overtopping does occur, they are can be highly susceptible to catastrophic failure unless overtopping duration and intensity are limited and erosion-resistant materials are used to construct the levee (or unless erosion protection, in the form of “armoring” is applied to the exposed crest and faces of the levee.) As water passes over the top of the levee, its velocity increases as it runs down the back side, and the erosive shear force of the water increases with this increase in velocity.

Accordingly, overtopping erosion is usually most severe initially low on the back slope face. Eventually, as the crest is “notched”, the resulting flow through the crest can also scour rapidly and further exacerbate the erosive process.

(b) Sharp Overtopping and Jetting – This is more likely to occur on levees with floodwalls. As shown in Figure 10.3, jetting occurs when the water level on the outboard side of the levee exceeds the top of wall elevation for structural walls that are founded within the earthen levee. Unlike overtopping of a conventional earthen levee, the floodwall acts as a weir and water falling over the wall impacts the levee in a concentrated stream that is much more energy intensive than conventional overtopping. For typical New Orleans floodwalls, the water impact velocities are on the order of 6 to 8 m/s. Levees are not generally designed for overtopping and jetting and as a result, if overtopping and jetting does occur, a deep scour trench can rapidly develop against the land-side face of the floodwall. This reduces the earth pressure providing lateral support for the wall, making the wall highly susceptible to potential catastrophic lateral failure (when pushed by the water pressures on its outboard side). This erosive trenching can be prevented by installing “splash-pads”, coarse rip-rap, or other energy dissipating devices at the inboard side toe of the floodwall, and the use of highly erosion-resistant materials to construct the crown section of the levee is also advisable here.

(c) Lateral Surface Erosion – As shown in Figure 10.4, lateral surface erosion generally occurs on the outboard side of the levee and is the result of water flowing past the levee face, or against the face of the stream channel banks below. If the imposed shear stress from the water abrading against the soil face is high enough, soil scour occurs and the integrity of the overall levee is significantly reduced. Levees that are exposed to chronic water flow, such as river levees, are generally designed and constructed with armoring or erosion protection to minimize scour-induced surface erosion. In general, well-compacted levees constructed of high-plasticity clays are much more resistant to surface erosion than uncompacted cohesionless soils (e.g. “clean” sands) and silty sands. Surface protection such as rip-rap, concrete pads, soil-cement reinforcement, and select vegetation coverings are typical methods used to protect levee faces from surface erosion.

(d) Wave Impacts – As shown in Figure 10.5, wave impacts can cause significant erosion to levee faces. Wave-induced erosion consists of run-up (sloshing up and down of water as a result of staggered wave arrival) and “mini-jetting” when the crest of the waves breaks on the levee face. Levees that are anticipated to be impacted by waves are generally designed with armoring to prevent damage from wave impacts.

The aforementioned failure mechanisms are not intended to be an exhaustive list, but rather to highlight common potential failure modes of interest for the levees in the New Orleans region.

10.3. Design Standards

Design standards are not just the primary means by which earthen levees are designed; they are also the main metric by which proposed levee design and construction projects are assessed and critiqued by reviewers. Incomplete, inaccurate, or inappropriate design standards can lead to actual field performance which is less than desired. As part of this study, current

earthen levee design standards from the United States Army Corps of Engineers and the United States Federal Emergency Management Administration were reviewed. A summary of the design guidelines for the USACE and FEMA are presented in Sections 10.4 and 10.5, respectively. This was not a fully exhaustive review (as that would have been beyond the scope of our current study), but this issue was studied as it provides important context for understanding some of the designs and decisions executed in the development of the pre-Katrina New Orleans regional flood protection system.

10.3.1. United States Army Corps of Engineers Design Standards

The primary manual and summary of design standards for earthen levees for the United States Army Corps of Engineers (USACE) is EM 1110-2-1913, "Engineering and Design – Design and Construction of Levees." This design manual covers the topics of: field investigations, laboratory testing, (fill) borrow areas, seepage control, slope design and settlement, levee construction, and special considerations (such as pipelines and other utility crossings, access roads and ramps, levee enlargements, junctions with concrete closure structures, and other special features such as landside ditch construction and levee vegetation management.)

10.3.1.1 Primary Design Procedure

The design procedure and requirements for levee design are established by EM 1110-2-1913. The outlined design procedure provides guidance from the initial preliminary evaluation through final design. These requirements (the principal "steps" in the design process) are summarized in Table 10.1.

Further engineering analysis guidance for the design of levees is provided in the following manuals:

- | | |
|-------------------------------|----------------|
| ▪ Slope Stability Analyses | EM 1110-2-1902 |
| ▪ Settlement Analyses | EM 1110-2-1904 |
| ▪ Levee/Structure Transitions | EM 1110-2-2502 |

The punch list of design steps identified in Table 10.1 provides an overview of design parameters and principal steps for levee design. EM 1110-2-1913 prescribes required Factors of Safety for slope stability of newly designed levees, existing levees, and other embankments and dikes. These Factors of Safety vary from 1.0 for short-term loading conditions to 1.4 for long-term (steady state conditions). Specific design criteria are not provided for settlement and erosion-susceptibility.

In addition to these design parameters, material specifications and construction procedures, critical elements in the actual life-span performance of levees, have also been defined by the USACE. These components are described in more detail in the following sections.

10.3.1.1. Material Selection

Acceptable soils for the construction of levees (borrow materials) are defined by EM 1110-2-1913 as “any soil is suitable for constructing levees, except very wet, fine-grained soils or highly organic soils.” Choosing a material type is generally a function of accessibility and proximity to the project area. The design guidelines emphasize that studies should be performed to ascertain the in-situ moisture contents of the borrow materials. It is noted that “the cost of drying borrow material to suitable water contents can be very high, in many cases exceeding the cost of longer haul distances to obtain material that can be placed without drying.” Thus, any materials may be used in the construction of levees so long as they are not overly wet fine-grained soils or highly organic soils. As will be discussed later in this chapter, our field observations and laboratory testing clearly show that a high-performance levee must be constructed from superior materials, and that utilization of more marginal materials (as allowed by the design guidelines) can result in catastrophically poor performance.

10.3.1.2 Required Levee Soil Compaction

Three general types of engineered earthen levees are presented in the EM 1110-2-1913 design criteria. These are compacted, semicompacted, and uncompacted levees. The USACE notes that, traditionally, compacted levees are usually used for areas of high property values and/or high land use, high populations, and for steep-sloped embankments with controlled compaction during construction which are utilized on good foundation conditions. Areas of low values, poor foundations, or high rainfall during the construction season generally warrant specification of semicompacted or uncompacted levees.

According to the USACE design guidelines, compacted levees are required to be constructed in areas where strong embankments of low compressibility are needed adjacent to concrete structures or forming parts of highway systems. Compacted levees require specification of appropriate ranges of fill material water content during compaction (with respect to standard effort optimum water content), initial loose lift thickness of typically 6 to 9 inches, compaction equipment type (e.g. sheepsfoot rollers, rubber-tired rollers, etc.), and either the number of compaction passes to attain a given percent compaction or standard maximum density or specification of the minimum required density (relative compaction).

Semicompacted levees are recommended by the USACE to be constructed in areas where there are no space limitations and thus steep-sloped embankments are not required, where onsite foundation soil conditions are relatively weak and unable to support steep-sloped embankments, where underseepage conditions require a wide base, and/or where the water content of borrow materials or rainfall during construction does not allow for the proper compaction of levee fill material. Semicompacted levees require the specification of lift thickness (typically approximately 12 inches) and are compacted by the movement of hauling and spreading equipment, or by sheepsfoot or rubber-tired roller compaction equipment.

The USACE recommends uncompacted levees only to be used for temporary/emergency use. These levees are constructed by fill cast or dumped in place as thick layers with little or no spreading or compaction. Hydraulic fill by dredge, often from channel excavations, is a common fill borrow source for uncompacted levees. Hydraulic fills are known to be highly susceptible to erosion upon overtopping and are not recommended to be used in the normal construction of

levees, except in locations where the levees are protecting agricultural areas whose failure would not endanger human life or for zoned embankments that include impervious seepage barriers.

10.3.1.3 Embankment Geometry

Embankment geometry specified by the USACE design guidelines is controlled either by material selection and compaction efforts during construction, or by the foundation soil conditions. Maximum side slopes for levees are 1V on 2H. These steep-sided levees are required to be constructed from high-grade borrow materials that are compacted near optimum moisture content and with appropriate compaction equipment. Levees with non-ideal borrow materials, such as sand levees, are required to have much shallower side slopes (on the order of 1V to 5H) to prevent damage from seepage and wave action. These design guidelines assume that this geometry and associated levee material will then be adequately resistant to scour and erosion, but as demonstrated by the numerous levee failures during Hurricane Katrina, this is not reliably always the case.

Final top of levee elevations must also account for future settlements, as determined by EM 1110-2-1904. In the past, the USACE specified a certain freeboard distance between the final top of levee elevation and the design storm water level to account for hydraulic, geotechnical, construction, operation, settlement and maintenance uncertainties. The updated design procedures set forth in EM 1110-2-1913 are risk-based, and are assumed to directly account for hydraulic uncertainties and establish a nominal level of protection.

10.3.1.4 Identified Potential Failure Modes for Design

The principal causes of potential levee failures, as identified by EM 1110-2-1913, consist of the following mechanisms (but see also Section 10.2 of this chapter):

- Overtopping;
- Surface erosion;
- Internal erosion (piping); and
- Slides within the levee embankment or the foundation soils.

Considerable discussion is presented in the design manual to mitigate effects of internal erosion/piping (see EM 1110-2-1913 Chapter 5 – *Seepage Control*). Guidance on overtopping, surface erosion, and slides within the levee embankment or the foundation soils is not well-developed in this design manual. However, guidance is provided for the augmentation of soil-cement protection applied to exposed slopes, susceptible to erosion.

10.3.1.5 Erosion Susceptibility

Although not directly addressed or identified in EM 1110-2-1913, general guidelines for erosion susceptibility of fine-grained cohesive soils are presented in EM 1110-2-1100 [Coastal Engineering Manual Part III], EM 1110-2-1100 [Coastal Engineering Manual Part VI], and “Channel Rehabilitation: Processes, Design, and Implementation,” (1999). These manuals provide insights on erosion and critical values of average overtopping discharges. They provide valuable design information for levees situated in coastal areas, and these design concepts should be applied to urban levees situated in coastal areas. Levee stretches such as the MRGO and the

Orleans East southeastern levees abut Lake Borgne (an extension of the Gulf of Mexico) and are susceptible to wave attack during strong storms.

For coastal grass covered sea-dikes and protected embankment seawalls, EM 1110-2-1100 indicates that no damage occurs for overtopping discharges of less than about 0.15 ft³/s per foot. Significant damage is expected for overtopping flows greater than 0.35 ft³/s per foot. The overtopping discharge flows were based on wave run-up exceeding the crest of the embankment or floodwall crest, and include estimated impact forces associated with the wave action impacting the embankment. These values were based on field studies conducted both in the United States and in the Netherlands. Discharge flow values are not based on sustained overtopping discharge as a result of the mean storm water level rising above the crest of the embankment or floodwall.

Maximum permissible velocities for flow within river and stream channels are summarized in USACE (1999), and are based on field research from 1915 to about 1926. Permissible velocities (for a canal type section with an average depth of 3 feet) are presented in Table 10.2.

Equivalent shear stress in this Table was calculated using the following equation (Munson et al, 1990):

$$\tau_w = K\rho V^2/2 \quad \text{[Equation 10.1]}$$

In this correlation, the shear stress (τ_w) imposed on the surface exposed to the water flow is a function of the surface roughness (K), the density of the fluid (ρ), and the velocity of the fluid (V). Based on this table, permissible water velocities vary between 1.5 ft/s for highly erosion susceptible materials to as much as 6 ft/s for highly erosion resistant materials. Correspondingly, allowable shear stresses vary from a low of 2.4 lb/ft² for highly erosion susceptible materials to as much as 36 lb/ft² for highly erosion resistant materials. Again, erosion plus jetting can lead to impact velocities with erosive potential about 3 to 4 times the maxima above. Although these design guidelines are available for use, they do not appear to have been incorporated into the design of the MRGO frontage and New Orleans East coastal levees fronting Lake Borgne.

10.3.2 United States Federal Emergency Management Agency (FEMA) Design Standards

Separate from the USACE levee design guidelines, design criteria for levee systems required by the United States Federal Emergency Management Agency (FEMA) are presented in the Title 44, Volume 1, Part 65 of the Code of Federal Regulations. These criteria establish the minimum standards to which levees must adhere in order to satisfy the 100-year level (referred to as the base flood) of protection mandated by FEMA. The main design criteria for FEMA approved levees are: freeboard, closures/transitions, embankment protection, embankment and foundation stability, settlement, interior drainage, and other specialty design criteria deemed appropriate by FEMA for unique situations.

10.3.2.1 Freeboard

Levees constructed adjacent to rivers are mandated to have a minimum freeboard of three feet above the water surface level of the base flood. In areas where the levee is constructed adjacent to structures, such as bridges, an additional one foot of freeboard is required extending 100 feet to either side of the structure. Levees constructed on the coast must have a minimum freeboard of one foot above the height of the calculated one percent wave or the maximum wave run-up (whichever is greater) associated with the 100 year still-water surge elevation. This category best fits some portions of the New Orleans Hurricane Protection System. Exceptions may be granted, based on site-specific engineering studies, but a freeboard of less than two feet is not deemed acceptable under any circumstance.

10.3.2.2 Closures

Closures refer to openings within the flood protection system. These closures can be for through traffic (such as railroad traffic which is frequently grade controlled and can not easily be diverted over levees), for pipeline crossings, or for maintenance purposes. FEMA requires all closures to be structural parts of the overall flood protection system during operation, and that they be designed in accordance with sound engineering practice.

10.3.2.3 Embankment Protection

Engineering analyses are required to be performed to demonstrate that no appreciable erosion of the levee embankment will occur during the base flood due to currents or waves, and that any anticipated erosion will not result in failure of the levee embankment or foundation either directly or indirectly through seepage or subsequent instability. Specific factors to be analyzed to determine the adequacy of embankment protection are: expected flow velocities (especially in constricted areas), expected wind and wave action, ice loading, impact of debris, slope protection techniques, duration of flooding at various stage and velocities, embankment and foundation materials, levee alignment, bends, transitions, and levee side slopes. The FEMA guidelines do not, however, provide guidance regarding acceptable performance criteria/standards for the identified embankment protection factors to be evaluated.

10.3.2.4 Embankment and Foundation Stability

Stability analyses for levee embankments are required to be submitted that demonstrate the adequacy of both short-term and long-term slope stability of flood protection levees. Stability analyses are required to include the expected seepage during the storm loading conditions, and to demonstrate that seepage into, beneath or through the embankment will not result in unacceptable stability performance. FEMA provides for the use of the USACE Case IV (as defined by EM 1110-2-1913, "Design and Construction of Levees") as an additionally acceptable engineering analysis method. The required factors for evaluation include: depth of flooding, duration of flooding, embankment geometry and length of seepage path at critical locations, embankment and foundation materials, embankment compaction, penetrations, other design factors affecting seepage (such as drainage layers), and other design factors affecting embankment and foundation stability (such as interior berms). The FEMA guidelines do not, however, provide quantitative guidance regarding acceptable performance criteria/standards for the identified stability modes to be evaluated.

10.3.2.5 Settlement

Once levees have been constructed to the specified crest elevation, their ability to provide the desired degree of flood protection against the base flood is often dependent to large extent on settlements due to time-dependent compression of the foundation materials beneath the levee. In order to demonstrate the adequacy of the crest elevation over the intended service life, FEMA requires that engineering analyses be submitted that assess the potential and magnitude of future losses of freeboard as a result of levee settlement, and demonstration that freeboard will be maintained within the minimum freeboard requirements for the duration of the intended levee service period. Detailed analysis procedures, such as those specified in the USACE EM 1110-2-1904, "Soil Mechanics Design – Settlement Analyses," are expected. The required factors for evaluation include: embankment loads, compressibility of embankment soils, compressibility of foundation soils, age of the levee system, and construction compaction methods. There are no specific provisions for regional subsidence, tectonic subsidence, nor for potential water level rise due to long-term climate change.

10.3.2.6 Interior Drainage

FEMA requires that the protected side of the flood protection system be capable of draining onsite water. An analysis is required to be submitted that identifies the source(s) of potential flooding, the extent of the flooded area, and, if the average depth of flooding is greater than one-foot, the water-surface elevation(s) of the base flood. The analysis is required to be based on the joint probability of interior and exterior flooding and the capacity of facilities (such as drainage lines and pumps) for draining interior floodwater.

10.3.2.7 Other Design Criteria

In areas where levee systems have relatively high vulnerabilities, or other unique situations, FEMA may require other design criteria and analyses be submitted for review and approval. The rationale for the requirement of additional analyses will be provided by FEMA. The review and subsequent evaluation standard of the analyses for the specified design criteria are to be based on "sound engineering practice."

10.3.2.8 Other FEMA Requirements

In order for the levee flood protection system to be recognized by FEMA as providing protection for the base flood, additional requirements, beyond the established design procedures and criteria, are required to be in place. Maintenance and operation plans are required to be submitted that detail how the flood protection system will be maintained and operated during its service period. In addition, FEMA has certification requirements which require that a registered professional engineer certify the levee design, and that certified as-built plans of the completed levee be submitted. Federal agencies with responsibility for levee design may also certify that the levee has been adequately designed and constructed to provide the desired degree of protection against the base flood.

10.4 Storm Surge and Wave Action During Hurricane Katrina

During Hurricane Katrina, the earthen levees were subjected to storm surges and wind generated wave action. Accurately determining the magnitude of these forces is reliant on numerical simulations and modeling with calibration from field data such as in-place instrumentation that recorded data during Hurricane Katrina as well as post-hurricane field assessments, such as high-water marks. The most reliable storm surge and wave action information collected and recorded during Hurricane Katrina was captured by instrumentation installed at select locations within the greater New Orleans area. However, the number of instrumentation locations was extremely limited, and as a result, little reliable storm surge and wave action information is available from this source. Many of the instruments were damaged during the storm and only partial records were collected.

Instruments that recorded useful data used to establish storm surge and wave action information were located at the following locations (IPET 2006):

- Lake Pontchartrain near 17th Street Canal (*hydrograph & wave characteristics*)
- Pump station #6 on the 17th Street Canal (*hydrograph*)
- Lake Pontchartrain at the Lakefront Airport (*camera-based hydrograph*)
- Inner Harbor Navigation Channel at I-10 (*hydrograph*)
- Inner Harbor Navigation Channel at the Lock (*hydrograph*)
- Gulf Intracoastal Waterway at I-510 (*hydrograph*)

A detailed review and reconstruction of the storm surge and wave action during Hurricane Katrina based on the data from the installed instruments, measured high water marks and interviews was completed by IPET (2006). The storm surge and wave action information presented by IPET was used in our performance evaluation of the levees. A discussion of the maximum storm surges is presented in the following section along with an overview of our field reconnaissance and levee condition survey and mapping.

10.5 Field Reconnaissance and Levee Condition Mapping with Regard to Levee Erosion

Field reconnaissance was a vital part to assessing and understanding the performance of the earthen levee flood protection systems. Multiple field visits were performed by the team to visually observe and evaluate the performance of the levee systems. The initial levee assessments occurred between September 29 and October 15, 2005. The principal purposes of these initial site investigations were to perform an initial survey of major damage areas, to perform initial forensic studies at the major failure (breach) sites and at nearby, more successful sites, and to note and record time sensitive data and observations before ongoing emergency repair operations obscured vital storm-related levee system performance information. The results of our initial observations and findings are presented in Seed et al., (2005).

Subsequent to the initial field reconnaissance and forensic studies, a series of field survey explorations have been performed to extend the initial condition surveys and to collect physical samples for testing to ascertain susceptibility to erosion. The variable performance of the earthen levee flood protection components during hurricane Katrina provide a unique learning

opportunity in that many of the levee system elements were overtopped, impacted by moving objects and debris (such as steel barges and fishing boats), and/or attacked by wind-generated waves. Some sections performed extremely well, while other sections performed poorly. As a result, there is a valuable opportunity to draw empirical lessons regarding the interactions between water and wave loadings, embankment and foundation soils and geometry, and performance.

Figure 10.6 shows the extents of the formal visual reconnaissance (the dashed black line) that was completed as part of our follow-on study on this issue. Due to access, schedule, and funding limitations, the Independent Levee Investigation Team was not able to complete a full and comprehensive survey of the entire greater New Orleans area for this element of our studies. Locations of noteworthy performance have been identified in the numbered boxes on Figure 10.6, and these are discussed in further detail below (refer to Figure 2.6 in Chapter 2 for a summary of design elevations for the flood protection system). Please note that these locations are intended only to represent typical findings and are not intended to summarize the complete performance of the overall flood protection system. In addition, the specified design flood protection system component crest elevations may not be the actual crest elevations at the time of hurricane Katrina's arrival due to factors such as incomplete staged construction, consolidation and settlement, regional subsidence, difficulties in correlating datums and elevation benchmarks, etc.

It is important to emphasize that accurately determining elevations in the greater New Orleans area is extremely complicated. Factors that exacerbate the problem include regional subsidence, localized consolidation settlement, progressive settlement of benchmarks used to establish regional datums, and the temporal variation in time of completion of individual project sections. A tremendous effort has been undertaken by the IPET team to "equalize" all the locations that are part of the New Orleans Flood Defense System and to merge the many component-specific elevations to one common project-specific elevation datum.

Following is a summary and discussion of levee performance at select locations along the perimeter of the New Orleans Flood Defense System sections traversed in this section of our studies (as shown in Figure 10.6.) A detailed and more comprehensive assessment of the as-reconstructed levee system is recommended to be conducted upon completion of the ongoing repairs and upgrades.

10.5.1 Location 1 – Lakefront Airport

Location 1 is situated near the intersection of Downman Road and Hayne Boulevard, south of the old Lakefront Airport. At this location, an earthen levee connects to a railroad bridge and vehicular underpass and an adjoining concrete floodgate structure, and the levee is situated parallel to an active railroad line. This is a highly complex "penetration" (where several access ways pass through or across the federal levee; including the rail line and the roadway), and a complex set of "transitions" where disparate flood protection elements abut each other and must perform well in combination. Unfortunately, this is one of numerous sites where these elements were not adequately detailed and coordinated, and performance was unacceptably poor and a breach occurred at this location.

High water marks, as reported by IPET (2006), at this location reached approximately Elevation +12 feet (MSL). The design elevation of the levee system at this location was Elevation +13.5 feet (NGVD29). Exact datum conversions in this area are not clearly established and are still under review by the IPET team, but the design elevation has been identified as Elevation +11.8 feet (MSL), resulting in some degree of overtopping at this location.

Storm-surge induced overtopping traveled through the low spot at this complex transition/penetration, which was the granular gravel ballast for the railroad line, and this flow eroded the railroad line embankment, which served as a transition levee between the concrete floodwall (design Elevation + 13.5 feet MSL = +11.8 feet NAVD 88-2004.65) and the earthen levee (design Elevation +14.5 feet MSL = +12.8 feet NAVD88-2004.65) shown in Figure 10.7. Figure 10.8 shows the location where overtopping occurred resulting in significant scour around the floodwall and Figure 10.9 provides a view across the railroad line where the railroad line embankment was eroded allowing for the terminus of the earthen levee to be scoured. Note that at the time of our visit, the railroad embankment had already been repaired by railroad personnel.

Performance factors of the levee system that impacted the performance of the flood protection components included the following: (1) unprotected high-permeability ballast (gravel) which allowed high water levels to seep through the gravel ballast and erode the supporting railroad embankment, (2) inadequate transition details between the flood protection components which allowed for low points to be exploited, and (3) the presence of embankment and levee materials that were not erosion resistant, resulting in scour as a result of overtopping. Without redesigning this transition area, future performance at this location (under similar or more severe storm surge conditions) is anticipated to be poor, and it will likely breach again.

10.5.2 Location 2 – Jahncke Pump Station Outfall

Location 2 is situated near the intersection of Hayne Boulevard and Jahncke Road, near Lake Pontchartrain. At this location a concrete outfall structure protrudes through the flood protection levee. High water marks, as reported by IPET (2006), at this location reached a maximum Elevation of +12 feet (NAVD88-2004.65). The design elevation of the levee system at this location was Elevation +14.5 feet (NGVD29). Exact datum conversions in this area are not clearly established and are still under review by the IPET team, but the design elevation has been identified as Elevation +12.8 feet (NAVD88-2004.65), resulting in a minor degree of overtopping at this location. Our field reconnaissance verified that minor overtopping occurred at this location, as can be seen in Figures 10.10 and 10.11.

Figure 10.10 provides an eastward looking view. Small patches can be seen on the levee crest where minor erosion occurred. Figure 10.11 presents a view of scour-related erosion behind the concrete outfall structure transition.

Performance factors of the levee system that impacted the performance of the flood protection components included the following: (1) placement of rip rap boulders along the Lake Pontchartrain margin which aided in damping wind-driven waves approaching the levee, (2) the presence of the active railroad line which also aided in damping wind-waves, and (3) utilization

of moderately erosion-resistant embankment materials (moderately compacted clayey soils and sandy clay soils.)

As shown in Figures 10.10 and 10.11, performance at this location was acceptable despite the moderate overtopping; only limited erosion occurred at select locations across the earthen levee crest and on the back face, and additional erosion occurred preferentially at several soil/structure interfaces. None of this erosion was sufficient as to result in a breach of the flood defenses at this location. The moderate erosion that did occur, however, suggests that more severe overtopping would likely be more problematic and might well cause full breaching if this section is not upgraded for erosion resistance.

10.5.3 Location 3 – Eastern Perimeter of New Orleans East

Location 3 is situated approximately 0.6 miles east of Highway 11 and approximately 1 mile north of Chef Menteur Highway (Hwy 90). In this vicinity, the flood protection system consists primarily of earthen levees that are protected by both low-lying swamplands and trees on both the outboard (water) side and the inboard (protected) side of the levee. High water estimates, as determined through numerical modeling by IPET (2006), suggest that the water at this location reached a maximum Elevation of approximately +16 feet (NAVD88-2004.65). The design elevation of the levee system at this location was Elevation +14.5 feet (MSL). Exact datum conversions in this area are not clearly established and are still under review by the IPET team, but the design elevation has been identified as Elevation +12.4 feet (NAVD88-2004.65), resulting in significant and relatively sustained overtopping at this location.

Figure 10.12 provides a southward looking view. The overall condition of the levees in this area is excellent and no observable damage or erosion was encountered. As part of local improvements after hurricane Katrina, an outfall access structure, near Hwy 90, was outfitted with a rock-gabion transition zone to minimize scour around the concrete access structure, as seen in Figure 10.13. This is an excellent idea, and should serve to further improve the performance of this detail in future events.

The excellent performance at this site was likely due to a number of factors, including: (1) the presence of low-lying swamp areas which aided in damping wind-waves approaching the levee, (2) the presence trees and shrubs outboard and inboard of the levee which also aided in damping wind-waves, and (3) utilization of moderately to highly erosion-resistant embankment materials.

10.5.4 Location 4 – Southeast Corner of New Orleans East

Location 4 is situated at the southeast corner of the New Orleans East polder. In this vicinity, the flood protection system consists primarily of earthen levees adjacent to the GIWW, fronting Lake Borgne. A small stretch of low-lying swamp protects the outboard side of the levees in this area, but it affords relatively little effective protection from wind-driven waves when “Lake” Borgne swells with storm surges from the Gulf.

High water marks, as determined by IPET (2006) using numerical simulations, suggest that water levels at this location reached a maximum Elevation of approximately +16 feet

(NAVD88-2004.65). The design elevation of the levee system at this location was Elevation +19 feet (MSL). Exact datum conversions in this area are not clearly established and are still under review by the IPET team. As discussed previously in Chapter 7, significant erosion and breaching occurred at this location, and this length of levee frontage was the single largest point of ingress for the floodwaters that eventually inundated the New Orleans East protected basin. The IPET studies have ascribed this massive erosion principally to overtopping, but it is the view of this investigation that considerable erosion also occurred due to wave action prior to full overtopping, and that through-levee seepage and underseepage may also have played a role at some locations (see Chapter 7).

Figure 10.14 shows one of a number of eroded zones or “slots” of the original levee that was breached and scoured as a result of storm-surge induced erosion during the hurricane. The levee embankment at this section was comprised largely of material dredged from the excavation of the adjacent GIWW channel, and large portions of this embankment appear to have had little resistance to erosion.

Figure 10.15 shows completed levee rehabilitation work at the southeast corner. At the time of our visit, construction activities had shifted approximately 1 mile west and consisted of belly-dump trucks placing borrow material which was being spread by bulldozers and track-walked. Dump trucks were also directed to travel over the newly placed levee, employing the semicompaction technique defined in EM 1110-2-1913.

Performance factors of the levee system that impacted the unacceptable performance of the earthen levees along this frontage included the following: (1) lack of slope protection (and crest protection) to minimize erosion due to both storm-driven waves and overtopping flow, (2) the site’s location adjacent to the “open” and relatively deep waters of Lake Borgne allowing for significant wind-driven waves to form and scour the flood side of the levee (and to notch the crest), (3) the lack of useful protection from outboard side swamp and/or cypress groves, etc., to reduce the energy and intensity of wind-driven waves, and (4) the utilization of embankment fill soils of variable erosion resistance (and permeability) so that both wind-driven wave erosion and through-flow erosion, as well as overtopping (wave splashover) erosion are all likely to have been active at this location.

This levee frontage performed disastrously poorly, and these factors that contributed to that unacceptable performance must be eliminated in the future.

10.5.5 Location 5 – Entergy Michoud Generating Plant

Location 5 is situated along the GIWW/MRGO shared channel, immediately beneath the Hwy 47/Paris Road bridge. In this vicinity, the flood protection system consists primarily of earthen levees. High water marks, as reported by IPET (2006), at this location reached a maximum Elevation of +16.3 feet (NAVD88-2004.65). The design elevation of the levee system at this location was Elevation +15 feet (MSL). Exact datum conversions in this area are not clearly established and are still under review by the IPET team, but the design elevation has been identified as Elevation +13.2 feet (NAVD88-2004.65), resulting in moderate to significant overtopping at this location.. Our field reconnaissance verified that overtopping in fact occurred at this location.

Figure 10.16 shows actual overtopping of the levee as captured by a security video camera at the Entergy Michoud Generating Plant during Hurricane Katrina. Figure 10.17 presents a post-Hurricane Katrina view of the same section of levee shown in Figure 10.16. Only minor damage occurred on the protected side, with a majority of the damage appearing to have resulted from wave reflection from the adjacent bridge abutment. Excepting the section where some minor erosion occurred on the inboard-side slope of the levee, this levee frontage for many hundreds of feet in each direction showed little evidence of damage from relatively sustained, moderate to significant overtopping flows. The overall condition of the levees in this area was good and no major damage was encountered.

The good performance of this embankment in the face of sustained moderate overtopping was likely due to several factors including: (1) utilization of moderately to highly erosion-resistant embankment materials (clay), and (2) the small fetch of the GIWW/MRGO channel at this location which limited the height of the wind-generated waves.

10.5.6 Location 6 – GIWW/MRGO Southern Shoreline Levee

Location 6 is situated along the ICWW/MRGO interchange beneath the Hwy 47/Paris Road bridge, on the opposite side of the channel (on the south bank) from Location 5. In this vicinity, the flood protection system consists primarily of earthen levees with a concrete floodwall beneath the bridge that connects the eastern and western levee segments. High water marks, as reported by IPET (2006), at this location reached a maximum Elevation of +16.3 feet (NAVD88-2004.65). The design elevation of the levee system at this location was Elevation +14 feet (MSL). Exact datum conversions in this area are not clearly established and are still under review by the IPET team, but the design elevation has been identified as Elevation +13.2 feet (NAVD88-2004.65), resulting in significant and likely relatively sustained overtopping at this location. Our field reconnaissance verified that overtopping occurred at this location.

The overall condition of the levees in this area was good and no major damage was encountered. The concrete floodwall constructed beneath the Hwy 47/Paris Road bridge, due to its top of wall elevation being lower than that of the neighboring earthen levees, acted as a weir during the high water period and “sucked” in nearby steel barges, as shown in Figure 10.18. Despite the collision impact of the barges with the concrete wall, the system performed well. Some scour-related damage was observed at the transition between the concrete flood wall and the earthen levee. Figure 10.19 presents an eastward looking view of the levee, just west of the washed up barges. East of the Hwy 47/Paris Road bridge, Figure 10.20 shows a gas processing barge that collided with the earthen levee. The impact did not result in significant damage to the levee.

The good performance of the levee system at this location was likely due in large part to the utilization of moderately to highly erosion-resistant embankment materials; these erosion-resistant (clayey) materials were also capable of absorbing impact loads from the barges, allowing the barges to come to rest on the levee without breaching it. The small fetch of the GIWW /MRGO canal at this location may also have limited the height of the wind-generated waves, thereby minimizing wave-induced erosion of the levee materials.

During a subsequent visit to this site in March of 2006, we observed that a concrete apron was being installed by the USACE around the transition between the concrete floodwall and earthen levee. This transition detail is anticipated to minimize future erosion at this transition area during a future severe event and reduce the risk of an erosion-induced full breach of the levee.

10.5.7 Location 7 – Bayou Bienvenue Control Structure

As shown in Figure 10.6, Location 7 is situated along the northern end of the MRGO levee frontage, near the northeast corner of the St. Bernard Parish protected basin. In this vicinity, the flood protection system consists primarily of earthen levees that connect, with a concrete control structure (with steel floodgates) across Bayou Bienvenue. High water marks, as reported by IPET (2006), at this location reached a maximum Elevation of +18.4 feet (NAVD88-2004.65). The intended eventual design elevation of the levee system at this location was Elevation +17.5 feet (MSL). Exact datum conversions in this area are not clearly established and are still under review by the IPET team, but the design elevation has been identified as Elevation +13.2 feet (NAVD88-2004.65). As discussed in Chapter 6 (see Section 6.2), this levee frontage was incomplete at the time of hurricane Katrina's arrival, as a final crest raising to offset consolidation-induced settlements has not yet been implemented. As a result, major overtopping occurred at this location.

Figure 10.21 shows an aerial photograph of the Bayou Bienvenue control structure. The floodwall to the north of the control structure performed very well, withstanding an impact load from a steel barge which became lodged atop the concrete flood wingwall attached to the concrete control structure.

The southern side of the control structure did not perform well; massive erosion and scour produced a major breach at the contact between the concrete control structure and the adjacent earthen levee. The southern side of the control structure was built using mainly spoils from the excavation of the adjacent MRGO channel that are more erosion-susceptible than the clays on the northern side of the control structure. It is important to note that both sides of this control structure were subjected to similar loading conditions and overtopping occurred on both sides and as a result, this site offers a unique example of the importance of erosion-resistant soil materials. In addition, the southern portion of the control structure abuts the abandoned Bayou Bienvenue channel, as shown in Figure 10.21. It is not conclusive whether the backfill materials into the abandoned channel impacted the performance of the control structure, but further investigation should be employed to determine the performance factors for this side of the control structure.

Figure 10.22 shows a close up view of the flood control gate structure that acted as a weir as the water overtopped the flood protection system. Significant scour and erosion was observed around the structure. Upon a follow up visit in March of 2006, splashpads had been installed behind the flood gate structure. In addition, the steel barge had been removed from the concrete control structure and a vast sea of rip-rap protection installed around the control structure. Figure 10.23 presents a picture of the installed splash pads and Figure 10.24 presents a view of the control structure with the barge removed and placement of rip-rap. Figure 10.25 presents a

schematic of the mapped scour around this concrete structure observed in October of 2005. Around the ends of the concrete wall, about 10 feet of soil have been eroded.

Factors of the levee system that impacted the performance of the flood protection components included the following: utilization of moderately to highly erosion-resistant embankment materials on the northern end of the control structure, utilization of moderately to highly erosion-susceptible embankment materials on the southern end of the control structure, and possible effects of the old Bayou Bienvenue channel abandonment backfill materials on the southern portion of the control structure. Future performance at this location under similar or more severe conditions is anticipated to be good for the northern half of the control structure and poor for the southern half of the control structure. Significant overtopping should be expected for larger storm surge events.

10.5.8 Location 8 – Mississippi River Gulf Outlet

Location 8 is situated along the western edge of the MRGO, south of the Bayou Bienvenue Control structure and north of the Bayou Dupre Control structure. In this vicinity, the flood protection system consists primarily of earthen levees constructed from excavated materials from the MRGO channel. High water marks, as reported by IPET (2006), at this location reached a maximum Elevation of approximately +18 feet (NAVD88-2004.65). The design elevation of the levee system at this location was Elevation +17.5 feet (MSL), however, reports indicated that these levees were not fully completed and had crest elevations that were 3 to 4 feet lower than the specified design elevation. In addition, exact datum conversions in this area are not clearly established and are still under review by the IPET team, but the design water level has been identified as Elevation +12.7 feet (NAVD88-2004.65). During Hurricane Katrina, moderate to major overtopping occurred at this location. Our field reconnaissance verified that moderate to major overtopping occurred.

Figure 10.26 shows a United States Geological Survey topographic map of the MRGO area. The identified “spoil area” corresponds to the zone of poor levee performance. Figure 10.27 shows aerial photography taken by NOAA in early September 2005 along the MRGO and shows severe erosion/breaches in the levee and barges that floated over the top of the levee and came to rest inside St. Bernard Parish after water elevations receded. Figure 10.28 shows close up aerial photographs of the severely eroded levees.

A bank erosion study was performed by the USACE (1988) that identified the presence of highly erosion-susceptible soils within the MRGO alignment. Merchant shipping traffic that traversed the MRGO created wake-induced waves and drawdown that were eroding the channel banks, resulting in the widening of the MRGO from an intended 650 feet to an actual average width of 1,500 feet, more than double the design width. Comments submitted from the Lower Mississippi Valley District on the report made the following comment in response to selecting the bank erosion mitigation alternative of decommissioning the MRGO:

The alternative to completely close the MRGO waterway should be evaluated....This alternative will control all future channel maintenance problems by controlling bank erosion, preventing the associated biological resource problems, preventing saltwater intrusion, and lessening the recreational losses.

In addition to solving the aforementioned problems, it will also reduce the possibility of catastrophic damage to urban areas by a hurricane surge coming up this waterway and also greatly reduce the need to operate (and could possibly eliminate) the control structures at Bayous Dupre and Bienvenue

Slope protection measures were recommended to aid in stabilizing these highly erosive deposits against wave-induced erosion. At the time of Hurricane Katrina, slope protection measures along the flood side of the MRGO levee were not in place. As identified in the above comments, the Hurricane Katrina storm-surge massively eroded the levees and resulted in catastrophic failure.

Performance factors of the levee system that impacted the performance of the flood protection components included the following: utilization of highly erosive embankment materials, lack of appropriate surface slope protection to minimize erosion of the flood side of the levee during the storm-surge, and as-constructed crest elevations below design elevations allowing for significantly higher water overtopping heights. Future performance, based on prior performance, at this location under similar or more severe conditions is anticipated to be poor unless improved materials and construction methods are used. We were unable to sample this location and test the materials for erodibility, but we did perform a follow up visual reconnaissance in April of 2005, and at that time we were very favorably impressed at the calibre of the work, and of the materials, being placed and compacted. During that follow-on reconnaissance, we did not detect the use of unsuitable fills, but we would like to return to sample and test the soils used to construct the levee in this section of the MRGO.

10.5.9 Location 9 – Bayou Dupre Control Structure

Location 9 is situated approximately 6.5 miles southeast of the Bayou Bienvenue Control structure, on the west side of the MRGO. In this vicinity, the flood protection system consists primarily of earthen levees that connect via a concrete and steel flood access structure to a concrete control structure across Bayou Dupre. High water marks, as reported by IPET (2006), at this location reached a maximum Elevation of +17 to +22 feet (NAVD88-2004.65). The design elevation of the levee system at this location was Elevation +17.5 feet (MSL). Exact datum conversions in this area are not clearly established and are still under review by the IPET team, but the design water elevation has been identified as Elevation +12.7 feet (NAVD88-2004.65), resulting in moderate to major overtopping at this location. Our field reconnaissance verified that moderate to major overtopping occurred.

Figures 10.29 and 10.30 show aerial photographs of the Bayou Dupree control structure. The area to the south of the control structure performed very well, while the northern side of the control structure did not perform well, with significant erosion and scour as a result of the overtopping. The northern portion of the control structure abuts the abandoned Bayou Dupre channel, as shown in Figure 10.31. It is not conclusive whether the backfill materials into the abandoned channel impacted the performance of the control structure, but further investigation should be employed to determine the performance factors for this side of the control structure.

Figures 10.32 and 10.33 show aerial photographs of repair operations underway at Bayou Dupre in January 2006. As can be seen in Figure 10.33, sand borrow material has been

imported to be used in the backfilling repair operations in the deep scour pools on the north side of the control structure.

Performance factors that impacted the performance of the flood protection components at this location included the following: utilization of highly erosive embankment materials, lack of appropriate surface slope protection to minimize erosion of the flood side of the levee during the storm-surge, as-constructed crest elevations below design elevations allowing for significantly higher water overtopping heights, and possible effects of the old Bayou Dupre channel abandonment backfill materials on the northern side of the concrete control structure.

10.5.10 Location 10 – St. Bernard Parish Interior Levee (Forty Arpent Levee)

Location 10 is situated north of the Corinne Canal and approximately $\frac{3}{4}$ miles east of Hwy 47/Paris Road. In this vicinity, the flood protection system was designed to be a secondary containment system for potential overtopping-related flooding behind the MRGO frontage levees and to act as a barrier against rainwater that is discharged into the swamp area. This part of the flood protection system consists primarily of earthen levees with a design Elevation of +8.0 to 9.0 feet (MSL). The actual elevation of this system during Hurricane Katrina was on the order of 5 to 8 feet (MSL) - (note that IPET did not establish NAVD88-2004.65 elevations at this location). High water marks were not reported by IPET at this location. Based on our field reconnaissance, it was apparent that major overtopping occurred at this location.

Figure 10.34 shows an eastward looking view of the earthen levee. Although this levee was significantly overtopped, it did not experience significant damage or erosion. Figure 10.35 shows a fishing boat that was washed over the levee shown previously in Figure 10.34 and came to rest in a residential neighborhood. The clearly excellent erosion resistance of this secondary levee, despite significant overtopping, was the result of use of cohesive, clayey soils as the levee fill material. The ability of these soils to sustain significant overtopping without catastrophic erosion is an important lesson.

Upon a subsequent visit to this location in March of 2006, we observed that the levee had been improved and raised by several feet to a new Elevation of +10 feet (MSL). Figure 10.36 presents the same view as in Figure 10.34, but 5 months later. Based on our observations, it appeared that cohesive soils and semi-compaction construction methods were used for this improvement. These soils appeared to be in a relatively erodeable condition as initially placed, but it also appeared likely that significant wetting from rainfall would improve their resistance to erosion from overtopping.

The performance factor that most significantly influenced the observed surprisingly good performance of the flood protection levee embankment at this site was the utilization of moderately to highly erosion-resistant embankment materials. Future performance of the levee embankment at this location under similar or more severe conditions is also anticipated to be good, however, significant overtopping should be expected for larger storm surge events which may cause the MRGO levees to breach, as this secondary levee has a crest elevation well below that of the main MRGO frontage levees.

10.5.11 Summary of Observed Performance Factors with Regard to Erosion Due to Overtopping

Based on observations from our field reconnaissance and review of aerial photographs, it is apparent that the performance and post Hurricane Katrina conditions of the earthen levee systems varied significantly, from good performance in areas with major overtopping to poor performance in areas with minor degrees of overtopping.

Table 10.3 presents a summary of the 10 locations evaluated as part of this study. Most of the levees studied in this pilot study were overtopped as a result of the large storm surge that rushed onshore. The magnitude of the storm surge and resulting overtopping did not, however, singularly dominate the observed performance of the levees. Additional factors were the degree of outboard side protection afforded by cypress swamps (which diminished wave energies), and the intrinsic resistance of the levee embankment and foundation soils to erosion.

Table 10.3 lists these factors, as well as the observed performance of the sections studied. As with the ILIT team's overall observations and studies of failures and successful performances throughout the New Orleans Flood Defense System, it is apparent that the use of highly erodeable soils such as cohesionless (sandy) soils represents a potentially unacceptably hazardous condition, and that the use of suitably compacted cohesive, clayey soils with relatively high intrinsic resistance to erosion can provide a measure of ductility and resilience to an otherwise brittle system (levees that can overtop for some number of hours, without catastrophically eroding and breaching.)

10.6 Erosion Susceptibility Evaluation

To date, the field of scour and erosion has not been well characterized and there is a paucity of well-defined field case studies that relate actual performance to design parameters. Both the USACE and FEMA design guidelines do not specifically provide acceptability criteria for erosion susceptibility due to the lack of comprehensive knowledge in this area. As a result of the NOFDS being "loaded to failure," it has provided an unfortunate opportunity to recognize lessons learned and improve our body of knowledge for the performance of levee flood protection devices.

It is important to note the magnitude of devastation caused to the regional flood protection system as a result of erosion. The levee system along the MRGO, which is the main protection mechanism for the 100,000+ citizens of St. Bernard Parish (and the Lower 9th Ward) against storm surges from the Gulf of Mexico, was catastrophically degraded as a result of erosion during Hurricane Katrina. Similarly catastrophic erosion occurred at the "sister" section of levee at the southeast corner of the New Orleans East protected basin. Figure 10.37 shows a comparison of two LIDAR surveys of the MRGO levee system at the Bayou Bienvenue control structure, near the intersection of the GIWW at the northeast corner of St. Bernard Parish. Effects of subsidence can be clearly seen in the elevation differences on the north side of the control structure and the control structure itself between 2000 and immediately following Hurricane Katrina in 2005. The levee on the north side of the Bayou Bienvenue control structure was largely undamaged. The levee on the south side of the control structure was catastrophically damaged. The magnitude of the erosion has been highlighted and white "splotches" of displaced levee materials can be seen in the aerial photograph.

There are many factors that influence the erosion susceptibility of soils, and a more comprehensive discussion of these was provided in Chapter 9. Fundamentally, soil erosion is controlled by the resisting characteristic of the soil (including soil type and character, soil fabric and structure, in-situ density, etc.) and eroding forces (including the magnitude and duration of the shear stress applied to the soil, impact or jetting pressures, etc.) from the contacting (eroding) fluid.

A sampling and laboratory testing program was devised upon completion of our field levee condition survey and mapping in October of 2005, to try to understand and characterize the properties associated with the levees that performed well and the characteristics of the levees that did not perform well during Hurricane Katrina. The intent of this study was to better understand the nature of the levee materials that performed well during the extreme conditions in order to provide recommendations on how to improve the sections of earthen levees that did not perform well.

In-situ samples were collected and from select levee sites during January and February of 2006. The selected sampling sites included levees that performed very well during Hurricane Katrina, levees that performed moderately well, and levees that did not perform satisfactorily. Figures 10.38 and 10.39 identify the locations where samples were collected for laboratory analyses.

Erosion susceptibilities of the soils were characterized using a state of the art erosion index testing method, developed by Dr. Jean-Louis Briaud at Texas A & M University, known as an Erosion Function Apparatus (EFA). This test method required undisturbed samples to be sampled from the field and be carefully transported back to Texas A&M University for analyses.

As described in Chapter 9, the EFA is a test that determines the shear stress and velocity of flowing water required to erode soil from a cylindrical tube that is slowly advanced into a rectangular pipe of flowing water. The more erosion resistant the soil, the faster the water (and the higher the shear stress) is required to flow in the rectangular pipe in order to erode the soil sample. A diagram of the EFA is presented in Figure 10.40. The measured shear stress at the point at which the soil begins to erode is defined as the critical shear stress. Shear stress less than the critical shear stress will not result in erosion, whereas applied shear stresses in excess of the critical shear stress will result in erosion. Determination of the erodibility index is useful in completing analyses for overtopping and surface erosion.

Upon completion of the test, the erodibility index of the soil was defined and the rate of erosion as a function of applied shear stress (or velocity) established. This relationship can then be compared with anticipated shear stresses the soil will experience in the field. If the estimated field shear stresses are less than the shear stress required to erode the soil, no erosion is anticipated to occur. If the field shear stresses exceed the laboratory determined critical shear stress, the erodibility index provides a means by which to estimate the magnitude of the overall erosion.

In addition to the erosion testing itself, additional engineering characteristics of the earthen levee sections under study were also characterized. These characteristics included the following:

- Gradation, including passing the Number 200 sieve (ASTM D422)
- Hydrometer (ASTM D422)
- Atterberg Limits Determination (ASTM D4318)
- Unconfined Compression (ASTM D2166)
- Dry density and moisture content determination (ASTM D4937/2216)
- Maximum dry density determination (ASTM D1557)

Table 10.4 presents a summary of the locations where samples were collected for analyses, and these locations are shown in Figures 10.38 and 10.39.

The samples were collected by pushing an approximately 3-inch diameter steel (Shelby) tube into the ground to retrieve soil samples using a geotechnical testing drill rig. Sites 4, 5, and 6 were located along the MRGO section of levee that suffered severe overtopping and erosion. At these sites the levee materials were collected in a soil sample bag and reconstituted back in the laboratory due to the highly disturbed nature of the levee materials.

The erosion susceptibilities of all the samples collected are presented in Figure 10.41. The test result designations are based on the Site Number, the boring number at the site, the depth interval over which the sample was collected, and additional sample notes. Thus, a sample marked as S1-B1-(0-2ft)-TW indicates that the sample came from Boring 1 at Site No. 1 (Levee east of HWY 11 and North of HWY 90) from a depth of 0 to 2 feet below the crest of the levee.

The results of the EFA test results are also presented in Table 10.4. The EFA test results matched very well with the observed performance in the field. Areas where the levee performance was observed to be good generally had low to very low erosion susceptibilities. In areas where the levee performance was poor, the materials had a high to very high erosion susceptibility.

The effects of material compaction were also evaluated. Previous work has been performed in this area and a design guideline prepared by FHWA (1988). Figure 10.42 shows that for a material of a given plasticity index, the permissible shear stress increases nearly ten-fold when the material is properly compacted.

Figure 10.43 shows the dramatic impact proper compaction can have on the erodibility of some types of soils. Materials sampled from the MRGO levee were tested at two compaction levels: low compactive effort and high compaction effort. The corresponding results speak volumes to the importance of compaction in earthen levees. The low-compaction sample was found to be very highly erodible, whereas the high-compaction sample exhibited low erodibility characteristics. High compaction effort is based on 90-95% relative compaction per the Modified Proctor test (ASTM D1557).

Figure 10.44 provides a summary from the pioneering erodibility work performed by Dr. Briaud et al. at Texas A & M University as part of these studies by (see Chapter 9). Soils that fell within the very high to high erodibility categories are prone to failure by overtopping. Soils

that fell within the medium erodibility category fell in a transition zone, and soils that fell within the low to very low erodibility categories were shown to be resistant to erosion induced failure as a result of overtopping. These laboratory test results were well-validated by actual levee performance during Hurricane Katrina.

10.7 Observed Failure Modes During Hurricane Katrina

Table 10.5 presents a summary of the types of failure mechanisms of earthen levees that were observed in the greater New Orleans area flood defense system, including the ten locations identified above. In addition, the required design evaluations per the USACE and FEMA guidelines have also been summarized in this table.

Overtopping, jetting, internal erosion and piping, underseepage and piping, underseepage-induced instability, and lateral and semi-rotational foundation instability failure mechanisms were all observed in the greater New Orleans region during Hurricane Katrina. There were multiple locations where trees that were rooted within the levee zone had fallen over and may have contributed to the failure of the levee. Liquefaction may have been a partial contributor to some of the failures along the MRGO levee system. The forces associated with the breaking waves impacting the MRGO levee may have been sufficient to induce liquefaction in the relatively weak foundation materials.

Some notable attributes were observed to be associated with levees that performed well during the hurricane. These attributes included:

- Utilization of erosion-resistant soils for levee construction;
- Gradual soil/structure transition zones (rock gabions around concrete structures);
- Presence of low-lying swamp and vegetation on the outboard sides to dampen wind-waves; and
- Presence of rip-rap protection.

Some notable attributes were observed to be associated with a number of levees that performed poorly during the hurricane. These attributes included:

- Utilization of low erosion-resistant construction material;
- Transitions between different flood protection component types;
- Lack of surface slope protection for erosion-susceptible soil levees; and
- Abandoned channel backfills underlying levees.

10.8 Establishment of Design Criteria and Acceptable Performance

Varying degrees of levee reliability and performance are required based on the assets that the levees are protecting. Early in the last century, most U.S. levees protected agricultural farm lands, where the consequences of levee breaching and subsequent flooding resulted primarily in the loss of crops. With the growth of urban areas into lowlands adjacent to rivers and coastlines

over the past two to three hundred years (the first urban levee was constructed in New Orleans in 1718), and especially over the past century, levees have become increasingly important defense mechanisms for industry, vital infrastructure elements such as drinking water, sewage, and electricity transmission, and, most importantly, large populous regions (cities and towns) and thus human life. Little guidance is provided by either the USACE or FEMA design criteria as to what are acceptable design standards for high-consequence urbanized areas vs. low-consequence agricultural areas.

The current approach to establishing design standards utilized by the USACE is conducting cost-benefit analyses. This financial/risk evaluation procedure analyzes the cost of achieving a certain level of protection and compares it with the recognized benefit associated with that level of protection. Unfortunately, the cost/benefit model used by the USACE for levee systems does not account for: (1) the loss of human life, (2) economic losses to cities, counties, and states as a result of a non-operational and non-functional revenue base (e.g.: businesses shut down due to damage and lack of utilities, lack of a work force to operate businesses, lack of a tax base due to the displaced residents, and lack of tourists, etc.), and (3) numerous other costs and losses known collectively as “secondary” and “tertiary” costs/losses associated with economic ripples that spread farther a field from the immediate locale of the disaster in question (as opposed to the more easily quantifiable primary or “direct” losses associated with system failure and consequent flooding.) These secondary and tertiary losses become increasingly important as the scope of a disastrous failure increases; these uncounted losses are most pronounced for full-blown “catastrophes.”

This is unfortunate, as it results in systematic under-valuation of the likely benefits of investing effort and resources to prevent disasters before they occur. In the case of hurricane Katrina, estimates of “losses” due to the catastrophic flooding of approximately 85% of the greater New Orleans region vary significantly at the time of this writing, but most independent estimates are on the order of ~\$100 to \$200 billion, and some estimates range as high as \$400 billion. A clear outlier is the recent estimate of damages proffered by the IPET study’s Draft Final Report (IPET; June 1, 2006) which estimated these damages at approximately \$25 billion. This was largely a function of the procedures which the USACE is required to employ in making potential loss projections; these systematically undercount expected losses, and they also undercounted actual losses in the wake of Katrina. This systematic under-evaluation of projected (and real) losses provides a poor basis for subsequent decision-makers (e.g. federal and local government) to base decisions regarding appropriate allocation of resources to defend against risk and threats.

10.8.1 USACE Risk Management Approach

In response to budget constraints, increased situations requiring cost-sharing, and general public concern for the performance and reliability of completed projects, the USACE has evaluated the use and methodology of risk-based analyses (especially as related to the geotechnical components of these projects). A seminar was convened in 1983 by the USACE in order to “incorporate more information into the safety assessment [of projects] than [traditional] factor of safety methods.” A more recent evaluation was undertaken, and the results of this effort are presented in Engineering Technical Letter (ETL) 1110-2-556, published on May 28,

1999. This study recognized that there are inherent uncertainties associated with infrastructure problems and that the total effect of risk and uncertainty on a project's economic viability should be examined in order for "conscious decisions" to be made reflecting "explicit trade-offs between risk and cost."

According to ETL 1110-2-556, major sources of uncertainty that require evaluation include the following:

- Uncertainty in loadings;
- Uncertainty in engineering analysis parameters;
- Uncertainty in analytical models (model bias);
- Uncertainty in performance;
- Conversion of empirically-derived performance modes;
- Frequency and magnitude of physical changes or failure events; and
- Conditions of unseen features.

ETL 1110-2-556 identifies special situations uniquely applicable to geotechnical problems that result in uncertainties with large magnitudes:

- Natural earthen materials generally exhibit high variability in composition and engineering properties;
- Engineering characteristics of soils can exhibit high variability due to composition, deposition, sampling, and field & laboratory testing procedures;
- Engineering analyses can be performed assuming total stress (excluding the effects of groundwater) or effective stress (including the effects of groundwater). As a result, groundwater uncertainties may either be included or excluded in the analyses;
- Consideration of spatial correlation of soil properties is required due to the variability of deposition history; and
- The spatial scale of the project (as much as tens of miles long for levees) requires "sectioning" of the system into subcomponents.

These uncertainty factors can result in very large ranges and broad distributions for parameter bounds. For example, a mean soil shear strength value, determined based on a subsurface field sampling and laboratory testing, which is used to evaluate the stability of levee slopes may naturally vary between $\pm 30\%$ of the mean value to as much as $\pm 75\%$ of the mean value. These broad distributions significantly impact the reliability of the resulting calculated answer.

The report summarized recommended target reliabilities for expected performance levels. These target reliabilities are presented in Table 10.6.

The approximate median Factors of Safety associated with the established expected performance levels were added to the target reliability indices presented in ETL 1110-2-556 according to the following formula:

$$F.S._{.50} = e^{(\beta \sigma \ln RS)} \quad \text{[Equation 10.2]}$$

In this formulation, β is the safety index, R is the capacity, and S is the loading/demand. A fairly typical coefficient of variation of 30% is assumed in both the loading and capacities, and a lognormal distribution for the system and the loads are assumed.

It is interesting to note that this approach had not yet been applied to the New Orleans regional flood defense system. Instead, the system had been designed using more “traditional” approaches, including the common use of a required Factor of Safety of $FS \geq 1.3$ for stability analyses associated with the short-term (“transient”) conditions produced by hurricane-induced storm surge, winds and waves.

If this is then compared with the projections of Table 10.6, the expected performance for these design criteria would be anticipated to be “unsatisfactory”, with an approximately estimated 7% probability of failure. It is, of course, a matter of judgement as to how many individual segments and intersections comprise the overall New Orleans regional flood protection system, and also how accurately the “typical” coefficients of variation for both loadings and resistances characterize each of these, but the overall accuracy of the projection of expected performance based on the simplified estimates of Table 10.6 could be argued to be well-justified by the multiple and catastrophic failures of the flood protection system that occurred during hurricane Katrina.

10.8.2 Other Risk-Based Approaches

Alternative approaches to establishing design levels exist. John Christian (2004) summarized studies that back-calculated the annual probability of failure based on failures of actual engineered systems by Baecher et al. The lives lost or financial losses associated with the failures were plotted against the back-calculated annual probabilities of failure. This plot provides a mechanism by which to ascertain the “targeted” level of performance (the targeted level of reliability) of a given engineered structure, based on historic practice in a number of diverse fields. The resulting plot is presented in Figure 10.45.

Additional examples include both the Netherlands and Hong Kong, which have risk-based decision making tools (Figure 10.46) as part of their planning process to establish acceptable levels of safety for engineered systems protecting significant populations based on the expected number of fatalities, as well as expected financial “losses” (Christian 2004). Failures that might impact large populations and may result in large numbers of fatalities (e.g.: greater than 1,000) are required to have very low annual probabilities of failure.

Using the risk management planning relationship (Figure 10.46) developed by the Hong Kong Government Planning Department as an example, a proposed engineered system that has the potential to result in 1,000 fatalities would have an acceptable risk (based on an annual frequency of occurrence) of 10^{-8} , the range over which the principle “As Low As Reasonably Prudent” (ALARP) reliability level is recommended varies from annualized Pf_{pa} of 10^{-8} to 10^{-6} , and a risk with a Pf_{pa} of less than 10^{-6} is considered unacceptable for any case.

Table 10.7 presents a summary of calculated Annual Return Periods (in years), based on the probability of occurrence limits recommended by the Hong Kong Government Planning Department. This example highlights the sociological decision made in Hong Kong that high consequence events should occur very infrequently, with an annual return period of 1 million years! Although this may not be realistic due to natural and anthropogenic uncertainties, the premise of varying acceptable risk as a function of consequences is rational and feasible.

The Dutch have well-developed risk-based approaches for targeting the reliability of flood protection systems. Like the southern Louisiana region, the Dutch face two distinct types of flooding risk; river floods and flooding from catastrophic North Sea storms and their associated storm surge, waves and winds. The Dutch lost a large fraction of their nation to ocean storm flooding in the mid 1950s, and determined to develop rationally risk-based flood defense systems to prevent recurrence of similar catastrophic flooding in the future. The levels of targeted flood defense levels, in terms of return periods, are interesting in contrast to the approximately 200 to 300-year recurrence level that was nominally targeted for New Orleans. The Dutch use a recurrence level of flood loading on the order of 1,000-10,000 years for river floods of populous areas (major towns and cities), and 10,000 years for ocean storms. For less populated (largely agricultural) areas, recurrence levels targeted for flood protection design are on the order of 500-1,000 years.

All of these targeted levels of protection greatly exceed the targeted levels for the New Orleans regional flood protection systems.

Another good way to look at targeted levels of reliability for flood protection is to re-examine Figure 10.45. This figure is re-plotted as Figure 10.47, with a red cross-hatched region showing approximately the level of reliability associated with the New Orleans regional flood protection system at the time of Katrina's arrival. Based on the studies of the ILIT team, it is inferred that the New Orleans flood defense system would have been expected to fail catastrophically about once every 50 to 100 years. The consequences of the failures during Katrina were on the order of 1,300 lives lost, and/or about \$100 to \$200 billion in economic losses.

A red dashed line indicating the approximate levels of reliability targeted by current U.S. practice with regard to dams is also shown in Figure 10.47. It is interesting to note that current U.S. practice for dams would have called for approximately three orders of magnitude higher level of reliability than that which was apparently provided by the New Orleans regional flood protection system (a factor of approximately 1,000 times higher reliability or assured safety.) That is largely because "dams" are generally assumed to be associated with large consequences in the event of failure. Few U.S. dams, however, have likely consequences of failure larger than those associated with the failure of the New Orleans regional flood defense systems during hurricane Katrina. This would appear to suggest that the flood defense systems for major metropolitan areas should be engineered to have targeted levels of reliability on a par with those commonly targeted for U.S. dams.

10.9 Brief Comments on Drainage and Pumping

As discussed in Chapter 4, the New Orleans area is situated below sea level and significantly below the river level in the Mississippi River. The proper functioning of the drainage and pumping system is critical to ensure the population of New Orleans is not inundated by groundwater, rainwater, or floodwater. Essentially every drop of rain that falls into New Orleans has to be pumped out, because most of the developed metropolitan and suburban regions are below mean sea level. In addition, underseepage constantly passes in beneath the perimeter levee systems, and has to be pumped out as well.

Accordingly, despite the potential critical flood-fighting contribution of the drainage and pumping systems, “the pumping stations have not been considered to be part of the hurricane protection system except in a few instances where the buildings are structural part of a levee or floodwall” (IPET, 2006). Based on the observed poor performance of the pumping systems during Hurricanes Katrina and Rita, this perception of the drainage and pumping system must be changed in order to provide more reliable system performance in the future.

There are about 80 pumping stations (IPET, 2006) in the four study areas, and a majority of these pumping stations are more than over 50 to 100 years old. The pump stations are powered by an assortment of electrical supply systems, and most of the older pump stations are powered by antiquated 25HZ power generation facilities constructed in the late 1910’s and early 1920’s. As a result of Hurricane Katrina, approximately one-third of the total system pumping capacity within the study areas was lost after the passing of the hurricane. Only 16% of the pumping stations were fully operational during the hurricane.

An extensive review of the performance of the pumping and drainage systems in the Jefferson, Orleans, Plaquemines, and St. Bernard parishes was beyond the scope of our ILIT studies. Such an evaluation was performed by IPET, however, and the results are presented in their draft final report (IPET; June 1, 2006). An overview map of the parishes and regions studied by this element of the IPET studies is presented in Figure 10.48.

Figure 10.49 presents a detailed view of pump stations within Jefferson parish. The maximum pumping capacity within Jefferson parish is 48,460 cubic feet per second (cfs) by a total of 27 pumping stations that drain an area of 73,500 acres (IPET, 2006). Figure 10.50 presents a detailed map of pump stations within Orleans parish. The maximum pumping capacity within Orleans parish is 48,900 cfs from a total of 23 pumping stations that drain an area of 60,000 acres (IPET, 2006). Figure 10.51 presents a detailed map of pump stations within Plaquemines parish. The maximum pumping capacity within Plaquemines parish is 12,065 cfs from a total of 21 pump stations that drain an area of 55,000 acres. Figure 10.52 presents a detailed map of pump stations within St. Bernard parish. The maximum pumping capacity within St. Bernard parish is 7,000 cfs from a total of 8 pump stations that drain an area of 17,620 acres.

IPET identified four major failure modes for pump station malfunctions during Hurricane Katrina. These consisted of (1) loss of operational staff as a result of evacuation orders at pumping stations that required manual operation, (2) loss of potable water to lubricate and cool pumps during operation as a result of municipal water distribution system malfunction, (3) loss

of electricity to power the pumping station, and (4) shut-down or disruption of the pump facilities as a result of flooding. Pumping failure from evacuations, flooding, loss of electricity, and loss of lubrication (potable water) accounted for 46%, 26%, 8%, and 4% loss of total pumping capacity, respectively (Figure 10.53). These failures and breakdowns do not reflect multiple failure modes, such as a pump station being evacuated, only later to be overwhelmed as a result of breaches in the flood defense system (i.e. a break in the drainage canal wall such as at 17th Street Canal).

In addition to these four failure modes, there are three other significant pump and drainage system-based failure modes that impacted the proper performance of the pump and drainage system during Hurricane Katrina. These failure modes are (1) reverse flow (Figure 10.54) through the pumping stations due to inadequate pump discharge elevation clearance (and a lack of internal reverse-flow protection), (2) loss of drainage capabilities as a result of breach of the drainage canal [such as within the 17th Street Drainage Canal, the Orleans Canal, and the London Avenue Drainage Canal as shown in Figure 10.55], and (3) lack of sufficient temporary storage capacity [such as within St. Bernard parish between the interior levee and the MRGO exterior levee], where discharged water is ponded behind the MRGO levee until the gates of the two control structures can be opened to allow the stored water to drain into Lake Borgne. It is interesting to note that water discharged from the Lower 9th Ward must drain through approximately 10.5 miles of bayou (Figure 10.55) until it is finally discharged from the protected area through the control structure gates, which are not configured to allow discharge of water during storm events, instead of having the pump station discharge directly into the much closer IHNC about 650 feet from the pump station.

Mitigation of these systemic flaws and performance inhibitors will require significant effort, and should prompt a full re-evaluation of the pumping and unwatering system configuration and details in order to develop an improved system that can function reliably both during and after major storm events.

10.10 Conclusions

Hurricane Katrina resulted in the catastrophic flooding of the greater New Orleans area. Although the magnitude of the storm surge that overwhelmed the levee flood defense system was greater than the capacity of the system, the extent of the devastating damage could have been greatly minimized if the system had been robustly designed. There were many miles of earthen levees that were significantly overtopped, but did not breach catastrophically. These levees that did not breach were only overtopped for a few hours' duration and the quantity of water that did flow over the levees could have been pumped out of the protected area utilizing the existing drainage network and pump infrastructure (see Chapter 4). The levees that were not able to withstand overtopping breached catastrophically, allowing the full magnitude of the storm surge to overwhelm the protected area.

Design guidelines need to be updated to ensure the design and construction of robust levee systems. All failure mechanisms must be acknowledged and included in the design evaluation. All levees should be designed to withstand overtopping. Material selection and compaction are critical components to ensure adequate performance and appropriate

specifications for material selection and compaction should be developed and should be incorporated into the design guidelines.

The current design guidelines sponsored by both the USACE and FEMA assume that overtopping does not occur and does not require safety in the event of overtopping of the levees.

A probabilistic approach should be utilized to determine the appropriate factor of safety for the design of these levee systems. Accounting for uncertainties in demands on the system (height of storm surges, wave impacts, etc.) as well as uncertainties in the capacity of the levee system (erosion resistance, foundation stability, etc.) must be included in the safety evaluation of the levee system.

The current design guidelines sponsored by both the USACE and FEMA are based on deterministic factor of safety levels that do not account for a broad range of uncertainties nor do they account for mechanisms to ensure an appropriate level of safety based on the consequences of failure.

10.11 References

- Christian, John T. (2004), "Geotechnical Engineering Reliability: How well do we know what we are doing?" *J. of Geotechnical and Geoenvironmental Engineering*, Vol. 130, No. 10, October 1, 2004.
- Google Earth, (2005), earth.google.com, <accessed: March 21, 2006>
- Hong Kong Government Planning Department. (1994). "Potentially hazardous installations." *Hong Kong planning standards and guidelines*, Chapter 11, Hong Kong.
- Interagency Performance Evaluation Task Force, (2006), "Performance Evaluation, Status and Interim Results, Report 2 of a Series, Performance Evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System," March 10, 2006.
- IPET, (2006), "Performance Evaluation of the New Orleans and Southwest Louisiana Hurricane Protection System, Draft Final Report of the Interagency Performance Evaluation Task Force, Volume VI – The Performance – Interior Drainage and Pumping," available online: <https://ipet.wes.army.mil/>, date accessed: June 1, 2006.
- Munson, Bruce R., Donald F. Young, and Theodore H. Okiishi, (1990), "Fundamentals of Fluid Dynamics – Third Edition," John Wiley and Sons, Toronto, Canada, ISBN 0-471-17024-0.
- Seed, Raymond B., et al., "Preliminary Report on the Performance of the New Orleans Levee Systems in Hurricane Katrina on August 29, 2005," Report No. UCB/CITRIS - 05/01, November 17, 2005.
- Topozone.com, (2006), www.topozone.com, <accessed March 21, 2006>
- United States Army Corps of Engineers, (2006), MRGO Levee Reconnaissance Handout, "MRGO Center Profile Line," April 8, 2006.
- United States Army Corps of Engineers, (2002), EM 1110-2-1100 (Part III), "Coastal Engineering Manual Part III," April 30, 2002.

- United States Army Corps of Engineers, (2002), EM 1110-2-1100 (Part VI), “Coastal Engineering Manual Part VI,” February 28, 2005.
- United States Army Corps of Engineers, (2003), EM 1110-2-1902, “Engineering and Design – Slope Stability,” October 31, 2003.
- United States Army Corps of Engineers, (1999), “Risk-Based Analysis in Geotechnical Engineering for Support of Planning Studies,” ETL 1110-2-556, May 28, 1999.
- United States Army Corps of Engineers, (1999), “Channel Rehabilitation: Processes, Design and Implementation,” July 1999.
- United States Army Corps of Engineers, (1990), EM 1110-2-1904, “Engineering and Design – Settlement Analyses,” September 30, 1990.
- United States Army Corps of Engineers, (2000), EM 1110-2-1913, “Engineering and Design – Design and Construction of Levees,” April 30, 2000.
- United States Army Corps of Engineers, (1989), EM 1110-2-2502, “Engineering and Design – Retaining and Floodwalls,” September 29, 1989.
- United States Army Corps of Engineers, (1988), “Mississippi River-Gulf Outlet, St. Bernard Parish, La., Bank Erosion, Reconnaissance Report,” February 1988.
- United States Army Corps of Engineers, (1983), “Proceedings, Seminar on Probabilistic Methods in Geotechnical Engineering,” Miscellaneous Paper GL-83-26, September 1983.
- United States Army Corps of Engineers, (1966), “Lake Pontchartrain, LA. and Vicinity, Chalmette Area Plan, Design Memorandum No. 3, General Design,” November, 1966.
- United States Code of Federal Regulations, 44CFR65.10, October 1, 2005.
- Versteeg, M. (1987) “External Safety Policy in the Netherlands: An approach to risk management,” *J. Hazard. Mater.*, 17, 215-221.

Table 10.1 Summary of Major Levee Design Steps

Step	Procedure
1	Conduct geologic study based on a thorough review of available data including analysis of aerial photographs. Initiate preliminary subsurface explorations.
2	Analyze preliminary exploration data and from this analysis establish preliminary soil profiles, borrow locations, and embankment sections.
3	Initiate final exploration to provide: <ul style="list-style-type: none"> a. Additional information on soil profiles b. Undisturbed strengths on foundation materials c. More detailed information on borrow areas and other required excavations
4	Using the information obtained in Step 3: <ul style="list-style-type: none"> a. Determine both embankment and foundation soil parameters and refine preliminary sections where needed, noting all possible problem areas. b. Compute rough quantities of suitable material and refine borrow area locations.
5	Divide the entire levee into reaches of similar foundation conditions, embankment height, and fill material and assign a typical trial section to each reach.
6	Analyze each trial section as needed for: <ul style="list-style-type: none"> a. Underseepage and through seepage. b. Slope stability. c. Settlement. d. Trafficability of the levee surface.
7	Design special treatment to preclude any problems as determined from Step 6. Determine surfacing requirements for the levee based on its expected future use.
8	Based on the results of Step 7, establish final sections for each reach.
9	Compute final fill quantities needed; determine final borrow area locations.
10	Design embankment slope protection.

Table 10.2 Permissible canal velocities with average flow depth of 3 feet

Material	Clear water, no detrius (ft/s)	Water transporting colloidal silts (ft/s)	Equivalent Shear Stress ¹ (lb/ft ²)
Fine sand (noncolloidal)	1.5	2.5	2.4 - 6.3
Sandy loam (noncolloidal)	1.75	2.5	3.1 - 6.3
Silt loam (noncolloidal)	2	3	4.0 - 9.0
Alluvial silt (noncolloidal)	2	3.5	4.0 - 12.3
Ordinary firm loam	2.5	3.5	6.3 - 12.3
Fine gravel	2.5	5	6.3 - 25.0
Stiff clay	3.75	5	14.1 - 25.0
Alluvial silt (colloidal)	3.75	5	14.1 - 25.0
Coarse gravel (noncolloidal)	4	6	16.0 - 36.0
Shales and hardpans	6	6	36.0

¹Assuming a roughness constant equal to 1 and fluid consisting of seawater.

Table 10.3: Performance summary for selected levee locations.

Location	Design Water Elevation (ft) ¹	Maximum Storm Surge Elevation (ft) ²	Overtopping	Immediate Post-Hurricane Levee Condition ²
1 - Lakefront Airport	[13.5] 11.8	12	Minor	Poor
2 - Jahncke Pump Station Outfall	[14.5] 12.8	12	Minor	Adequate
3 - Eastern Perimeter of New Orleans East	[14.5] 12.4	~18?	Moderate to Major	Very Good
4 - Southeast Corner of New Orleans East	[19.0] 13.0	~18?	Moderate to Major	Poor
5 - Entergy Michoud Generating Plant	[15.0] 13.2	16	Moderate to Major	Good
6 - IWW/MRGO Southern Levee	[14.0] 13.2	16	Moderate to Major	Good
7 - Bayou Bienvenue Control Structure	[17.5] 13.2	18	Moderate to Major	Good/Poor
8 - Mississippi River Gulf Outlet	[17.5] 12.7 (~10 ³)	17-22	Major	Poor
9 - Bayou Dupre Control Structure	[17.5] 12.7 (~10 ³)	17-22	Major	Poor
10 - St. Bernard Parish Interior Levee (Forty Arpent Levee)	[8] ~6 ⁴ (~3 ³)	Not Established	Major	Very Good

¹Elevations converted from NGVD29 elevation (in brackets) to equivalent NAVD88(2004.65) elevation, from IPET (2006)

²Based on NAVD88(2004.65) vertical datum. From IPET (2006)

³Elevation at the time of Hurricane Katrina was below the design elevation

⁴A conversion between the original design elevation from the NGVD29 to the new NAVD88(2004.65) elevation was not available from IPET

²This is an assessment of conditions immediately after the hurricane, before significant repair and reconstruction.

Table 10.4 Summary of sampling locations for laboratory testing

Site No.	Description	Latitude (°N)	Longitude (°W)	Erosion Performance
1	Levee east of Hwy 11 and North of Hwy 90	30.0895	89.8587	Good
2	Entergy Powerplant	30.0065	89.9389	Good
3	MRGO North Control Structure (North)	29.9996	89.9170	Good
4	MRGO Levee (northern section)	Not Established		Poor
5	MRGO Levee (middle section)	Not Established		Poor
6	MRGO Levee (southern section)	Not Established		Poor
7	St. Bernard Parish South	29.8769	89.7818	Good
8	St. Bernard Parish North	29.9558	89.9466	Good
9	Lakefront Airport Transition Levee	30.03344	90.026	Moderate
10	Hayne Blvd	30.05908	89.96697	Good Not
11	Hayne Blvd and Paris Road (Beach)	30.07577	89.94467	Applicable
12	Orleans East Southeast RR Transition	30.06156	89.83352	Poor
13	Orleans East Southeast Corner	30.04481	89.83089	Poor
14	Intracoastal Waterway North (New Levee)	30.03542	89.85399	Unknown
15	Intracoastal Waterway North (Remaining Levee)	30.02707	89.87448	Poor
16	Levee west of Entergy Plant	30.00465	89.95062	Good
17	St. Bernard Parish (Middle)	29.92541	89.8948	Good

Note: Geographical coordinates based on WGS84 datum.

(Table 10.4, Continued) Summary of sampling locations for laboratory testing

Site No.	Description	Performance	EFA Erosion Susceptibility Determination
1	Levee east of Hwy 11 and North of Hwy 90	Good	Low (IV)
2	Entergy Powerplant	Good	Low to Very Low (IV-V)
3	MRGO North Control Structure (North)	Good	Low to Very Low (IV-V)
4	MRGO Levee (northern section)	Poor	High (II)
5	MRGO Levee (middle section)	Poor	High (II)
6	MRGO Levee (southern section)	Poor	High (II)
7	St. Bernard Parish South	Good	Medium (III)
8	St. Bernard Parish North	Good	Medium (III)
9	<i>Lakefront Airport Transition Levee</i>	<i>Moderate</i>	<i>Not Tested</i>
10	<i>Hayne Blvd</i>	<i>Good</i>	<i>Not Tested</i>
11	Hayne Blvd and Paris Road (Beach)	Not Applicable	Not Applicable
12	Orleans East Southeast RR Transition	Poor	High to Medium (II-III)
13	<i>Orleans East Southeast Corner</i>	<i>Poor</i>	<i>Not Tested</i>
14	<i>Intracoastal Waterway North (New Levee)</i>	<i>Unknown</i>	<i>Not Tested</i>
15	Intracoastal Waterway North (Remaining Levee)	Poor	Very High to High (I-II)
16	<i>Levee west of Entergy Plant</i>	<i>Good</i>	<i>Not Tested</i>
17	<i>St. Bernard Parish (Middle)</i>	<i>Good</i>	<i>Not Tested</i>

Table 10.5 Levee Failure Mechanisms

Failure Mechanism	USACE Guidelines	FEMA Guidelines	Observed in Greater New Orleans Area
Overtopping	Not allowed	Not allowed	Yes
Jetting	Not allowed	Not allowed	Yes
Internal Erosion and Piping	Design criteria provided	Analyses required	Yes
Lateral Surface Erosion	Protection required	Protection required	Possibly
Wave Impacts	Protection required	Protection required	Yes
Structural Impacts	Not addressed	Not addressed	Yes
Slope Failures	Design criteria provided	Analyses required	Yes
Sliding	Design criteria provided	Analyses required	Yes
Underseepage	Design criteria provided	Analyses required	Yes
Liquefaction	Not directly addressed	Not directly addressed	Possibly
Bottom Heave/Blowout	Not directly addressed	Not directly addressed	Yes

Table 10.6 Target Reliability Indices

Expected Performance Level	Beta (β)	Probability of Unsatisfactory Performance	<i>Approximate Median Factor of Safety¹ (F.S.₅₀)</i>
High	5.0	0.0000003	2.5
Good	4.0	0.00003	2.1
Above average	3.0	0.001	1.7
Below average	2.5	0.006	1.6
Poor	2.0	0.023	1.4
Unsatisfactory	1.5	0.07	1.3
Hazardous	1.0	0.16	1.2

Table 10.7 Risk Levels for a System with the Potential for 1,000 Fatalities

Risk Level	Pf_{pa}	Annual Return Period (yrs)
Acceptable	$<10^{-8}$	$>100,000,000$
ALARP	10^{-6} to 10^{-8}	1,000,000 to 100,000,000
Unacceptable	$>10^{-6}$	$< 1,000,000$

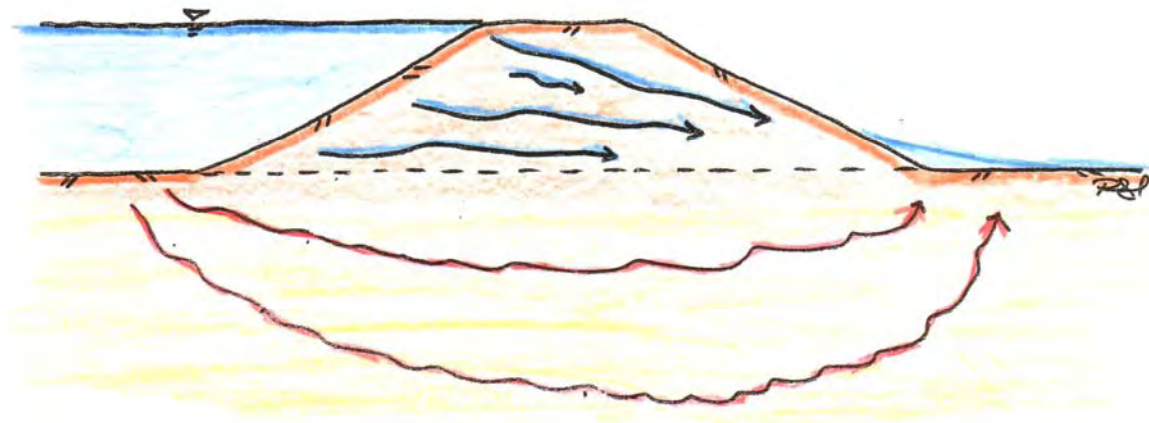


Figure 10.1: Underseepage in highly permeable underlying foundation materials (red lines) can result in the catastrophic failure of the levee in that once the foundation materials have been eroded, the levee (which may be completely undamaged) has no underlying support and falls into the resulting void and essentially washes away.

Internal erosion and piping (blue lines) occurs in levee materials that have high permeabilities (such as sand and gravel) and allow for water to rapidly flow from high pressure areas to low pressure areas. As the water flows through the levee, smaller/finer soil particles are “washed” out of the levee resulting in the internal erosion of the levee. Enough internal erosion of the levee can lead to the collapse and subsequent “wash-out” of the levee.

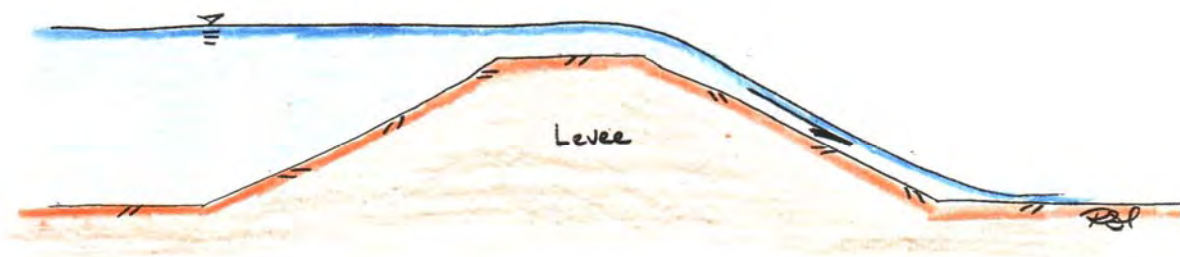


Figure 10.2: Overtopping occurs when the water level on the outboard side of the levee exceeds the crest elevation of the levee. The inboard side of the levee acts as a spillway for the overtopping water and damage is inflicted on the levee as a result of water scour.

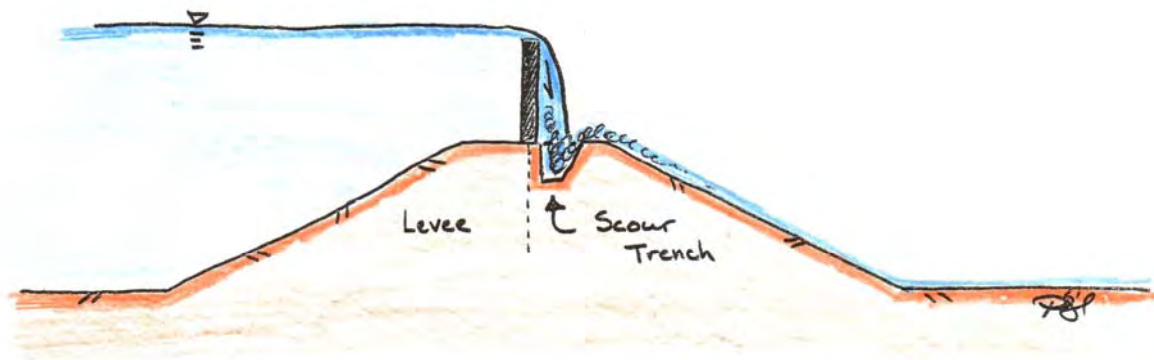


Figure 10.3: Jetting occurs when the water level on the outboard side of the levee exceeds the top of wall elevation for structural walls that are founded within the earthen levee. Unlike overtopping of a conventional earthen levee, the floodwall acts as a weir and water impacts the levee in a concentrated stream that is much more energy intensive than conventional overtopping.

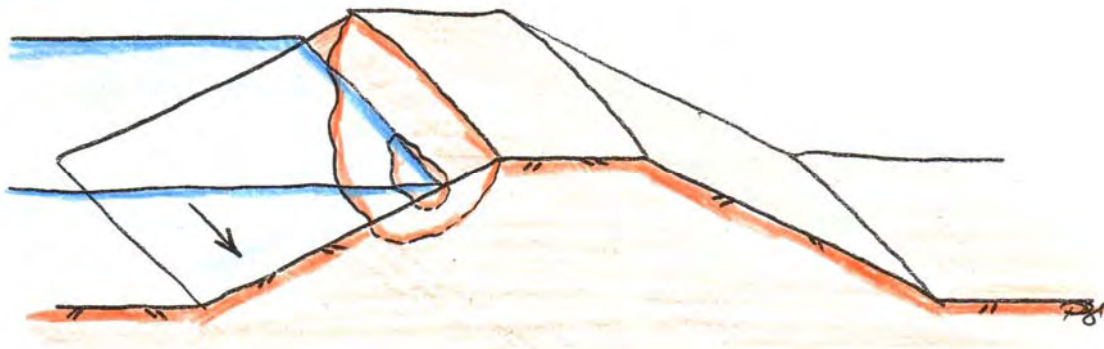


Figure 10.4: Surface erosion generally occurs on the outboard side of the levee and is the result of water flowing past the levee face. If the imposed shear stress from the water abrading against the soil levee face is high enough, soil scour occurs and the integrity of the overall levee is significantly reduced.



Figure 10.5: Wave impacts can cause significant erosion to levee faces. Wave-induced erosion consists of run-up (sloshing up and down of water as a result of staggered wave arrival) and “mini-jetting” when the crest of the waves breaks on the levee face.



Figure 10.6: Map showing the extents of the visual reconnaissance (dashed black line) of the earthen levee systems performed between October 2005 and March 2006. Locations of notable performance are identified in the numbered boxes.

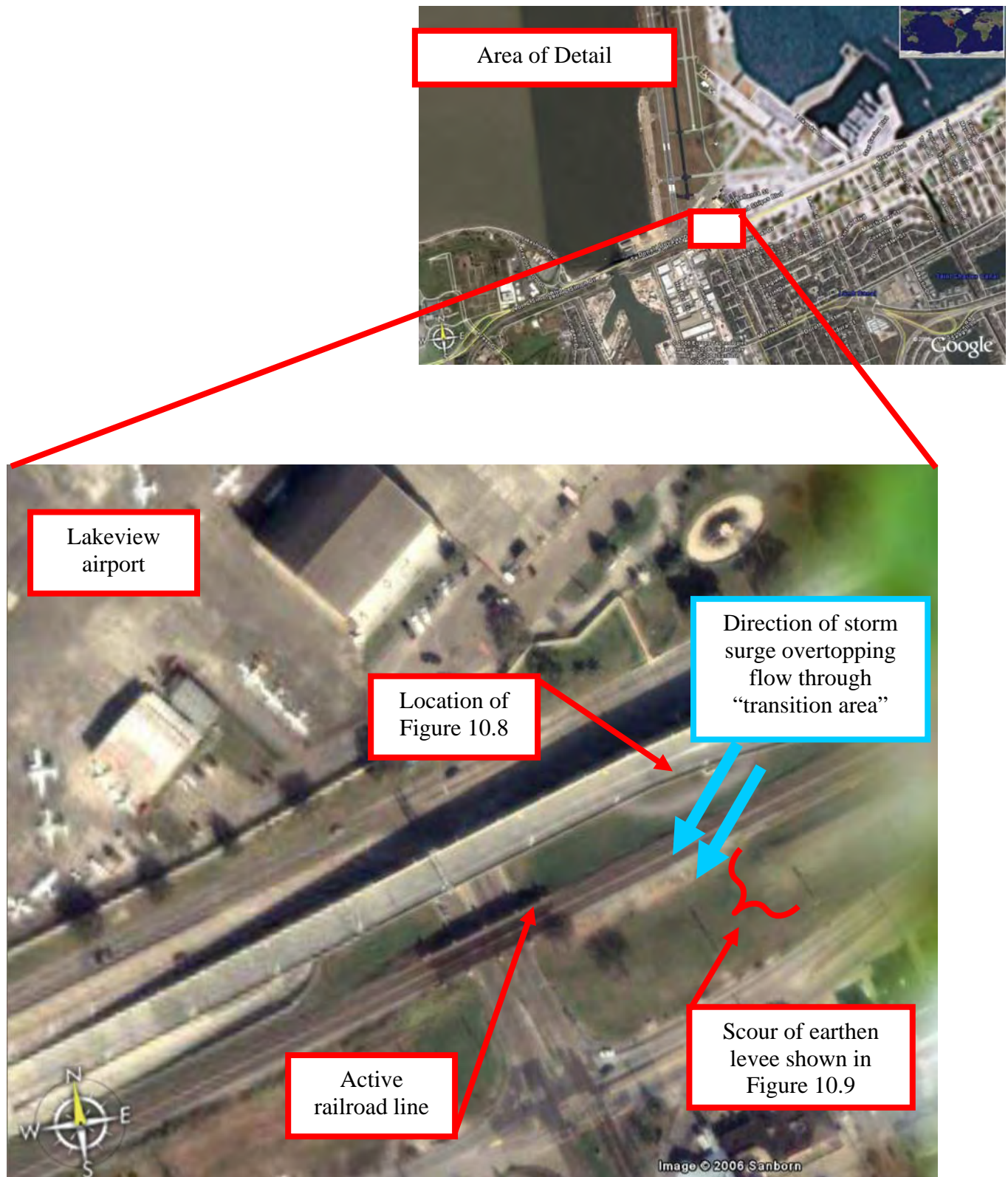


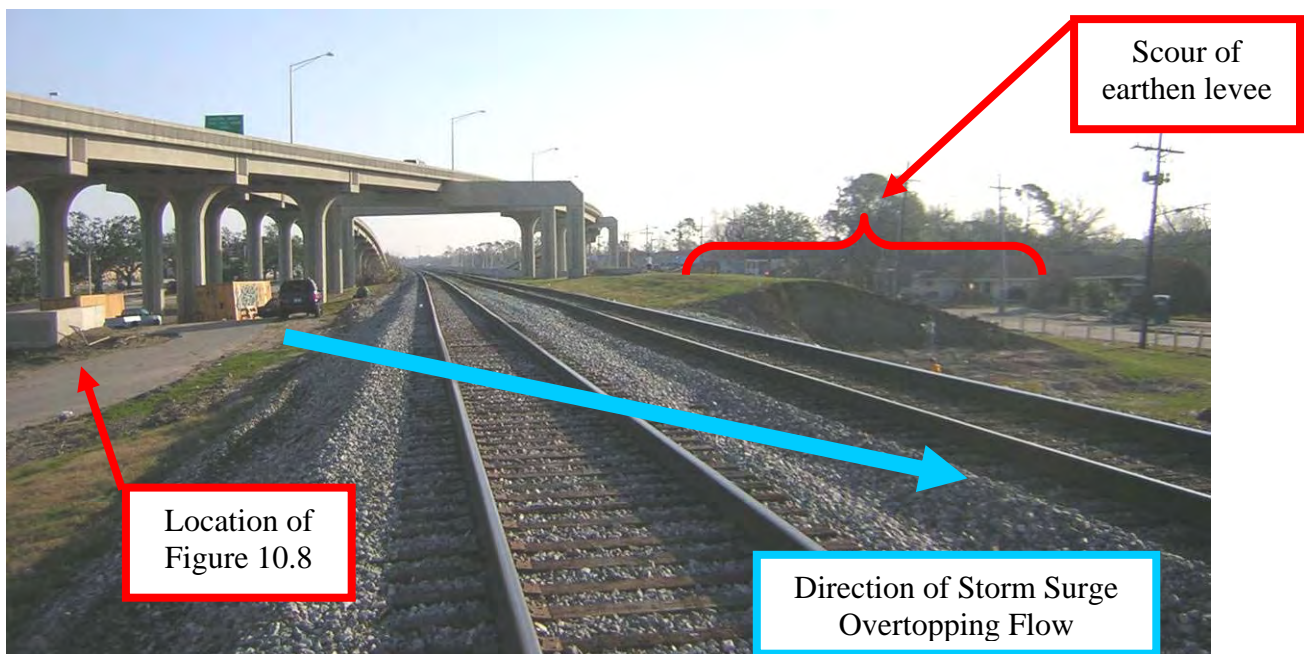
Image from Google Earth, 2006

Figure 10.7: Storm-surge induced overtopping traveled through the granular gravel ballast for the railroad line and eroded the railroad line embankment, which served as a transition levee between the concrete floodwall and the earthen levee shown in Figure 10.8.



Photograph by Rune Storesund

Figure 10.8: Lakefront levee near the Lakefront Airport at location 1 (as indicated on Figure 10.7) where overtopping occurred and significant scour around the floodwall was observed.



Photograph by Rune Storesund

Figure 10.9: Significant erosion was observed on the levee behind the floodwall shown in Figure 10.8. The storm surge overtopped the floodwall and railroad ballast and failed the earthen levee.



Photograph by Rune Storesund

Figure 10.10: Lakefront levee at location 2 (as indicated on Figure 10.6) where minor overtopping occurred. These levees performed well and only minor, surficial damage was observed.



Photograph by Rune Storesund

Figure 10.11: Observed scour at the Jahncke Pump Station outfall structure (location 2 as indicated on Figure 10.6). Scour was limited to areas of soil-structure interfaces, and no full breach occurred.



Photograph by Rune Storesund

Figure 10.12: Condition of levees east of HWY 11 (location 3 on Figure 10.6) in October 2005. These levees performed exceptionally well and were not eroded during Hurricanes Katrina or Rita.



Photograph by Rune Storesund

Figure 10.13: Levee rehabilitation work (near location 3 on Figure 10.6) after Hurricane Katrina included reinforcement and protection of soil-structure interactions with rock-gabion transition zones.



Photograph by Rune Storesund

Figure 10.14: Zones of earthen levees at the southeast corner of the New Orleans East polder (location 4 on Figure 10.6) were washed out during Hurricane Katrina allowing water to rush in.



Photograph by Rune Storesund

Figure 10.15: Levee rehabilitation work (location 4 on Figure 10.6) post Hurricane Katrina. The “semicompaction” construction approach was employed, utilizing earth moving equipment to also compact the placed material as it transported borrow materials to the construction site.



Photograph courtesy the Entergy Corporation

Figure 10.16: Photographs (location 5 on Figure 10.6) captured by a security camera at the Entergy Michoud Generating Plant beneath Route 47, on the IWW/MRGO show active overtopping during Hurricane Katrina.



Photograph by Rune Storesund

Figure 10.17: Post Hurricane Katrina view of the earthen levee shown in Figure 10.16. Only minor damage on the protected side was observed, with the majority of the damage a result of wave reflection from the bridge abutment at the right-hand side of the picture.



Photograph courtesy ngs.woc.noaa.gov/Katrina/KATRINA0000.HTM

Figure 10.18: The presence of a concrete floodwall beneath the Hwy 47/Parish Road Bridge acted as a weir during the overtopping stages of the storm and “sucked” in nearby barges. Scour can be observed behind the concrete floodwall, but there is only minor scour damage visible at the earthen levee/concrete floodwall transition. Overall, this system performed well (also considering the impact associated with the barges).



Photograph courtesy Lee Wooten

Figure 10.19: Woody debris and steel barges “washed up” on the southern ICWW/MRGO levee just west of Route 47 (location 6 on Figure 10.6). This is a side view from the aerial photograph presented in Figure 9.24.



Photograph courtesy Francisco Silva-Tulla

Figure 10.20: A gas processing barge “washed up” on the southern IWW/MRGO levee just east of Route 47 (location 6 on Figure 10.6). This levee was also overtopped and was not significantly damaged by the barge impact.



Photograph courtesy Les Harder

Figure 10.21: Aerial photograph of the Bayou Bienvenue Control Structure (location 7 on Figure 10.6). The northwestern half of the control structure levee system performed extremely well (withstanding significant impact loads from the steel barge), while the southeastern portion suffered severe erosion.



Photograph by Rune Storesund

Figure 10.22: This flood control gate acted like a weir as water overtopped the structure during the storm surge. Significant scour and erosion was observed around the flood control gate structure.



Photograph by Rune Storesund

Figure 10.23: By March of 2006, after Hurricane Katrina, splash pads had been installed at this flood control gate to mitigate erosion impacts if any future overtopping of the flood protection system occurs. This photograph was taken at the same location as Figure 10.22.



Photograph by Rune Storesund

Figure 10.24: The northern side of the Bayou Bienvenue control structure has been repaired, and then heavily reinforced with new rip-rap transported to New Orleans from Kentucky. This photograph was taken about 6 months after Hurricane Katrina and the barge, which can be seen in Figure 10.21, has been removed.

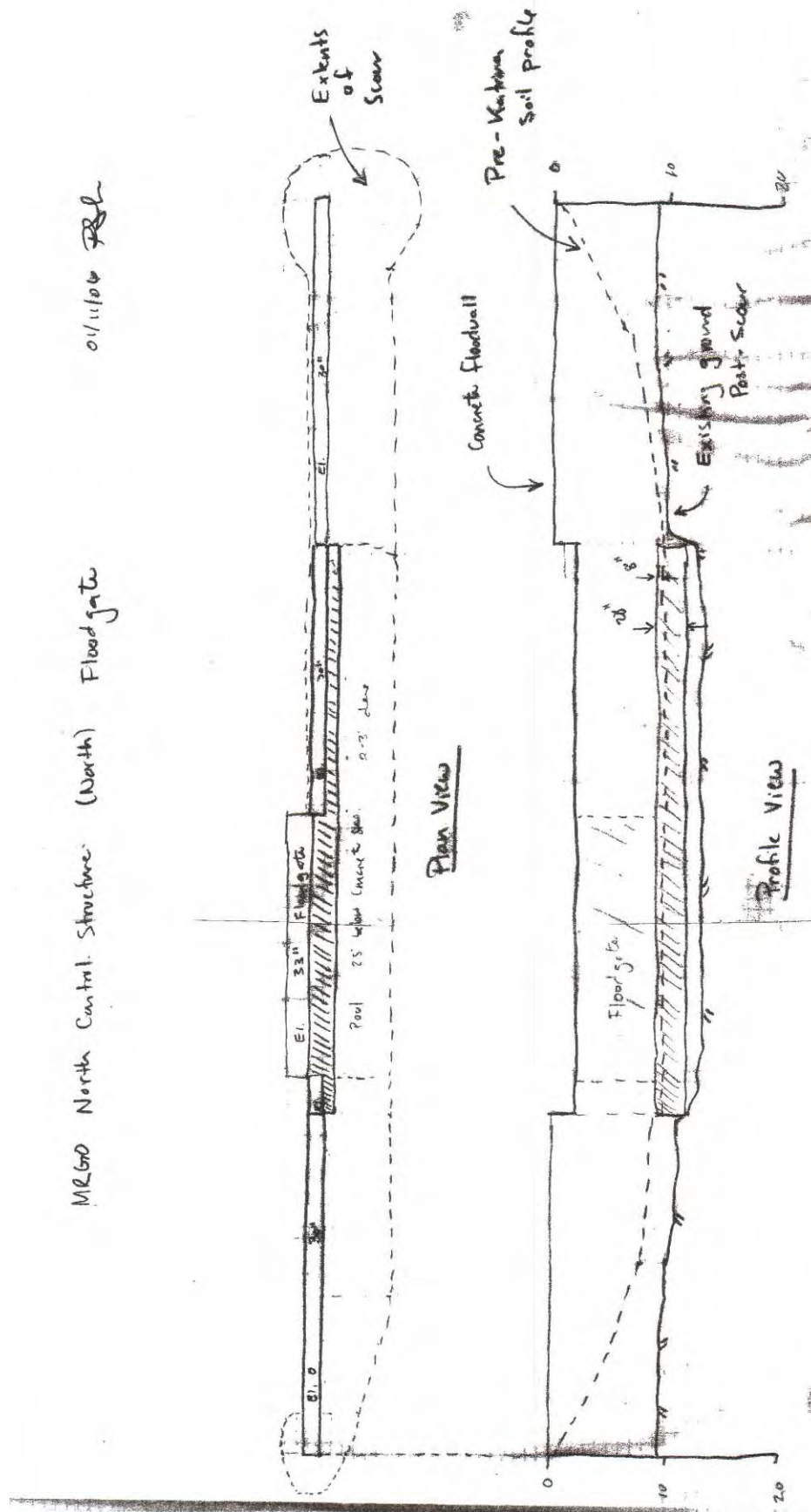
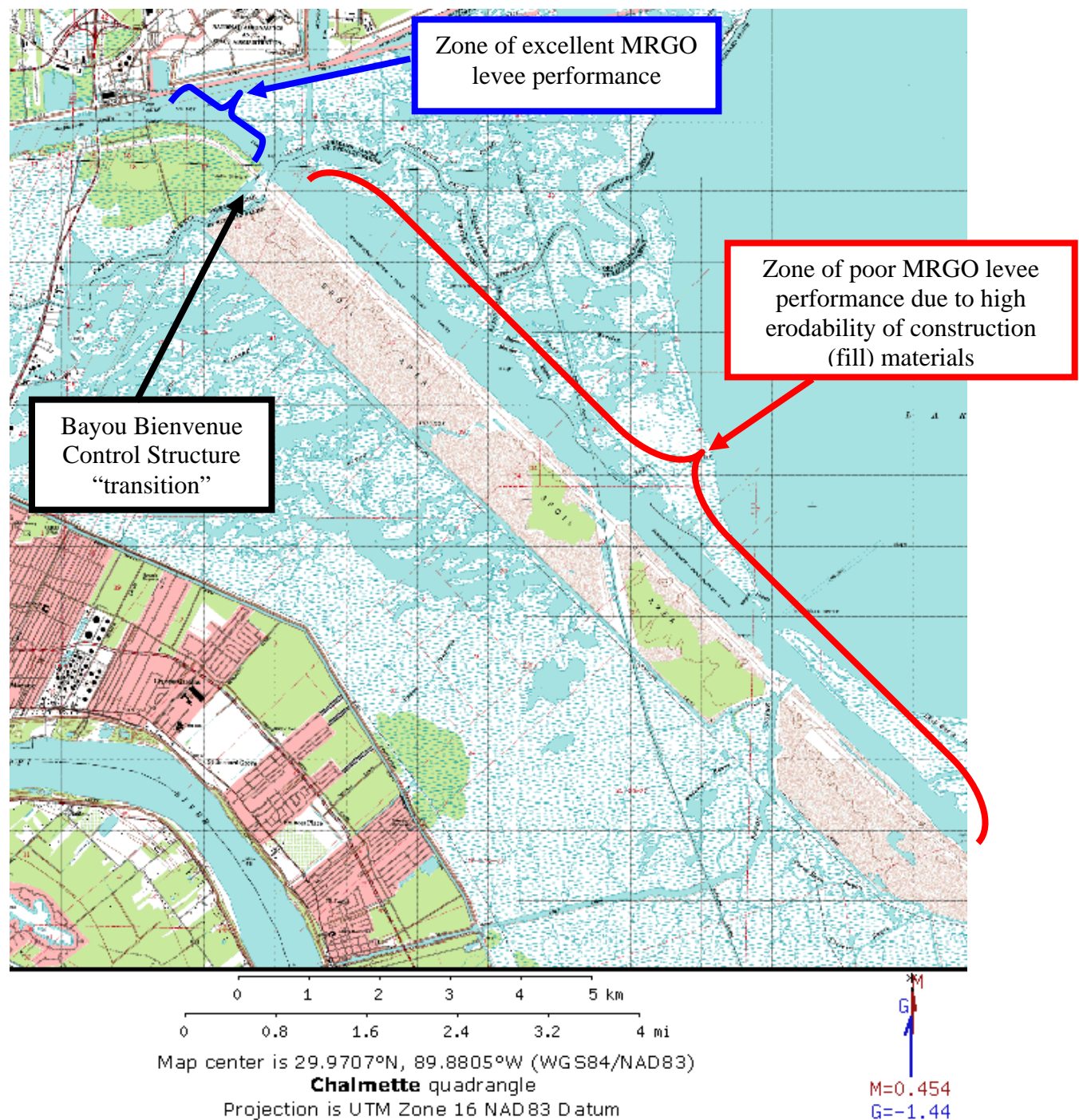
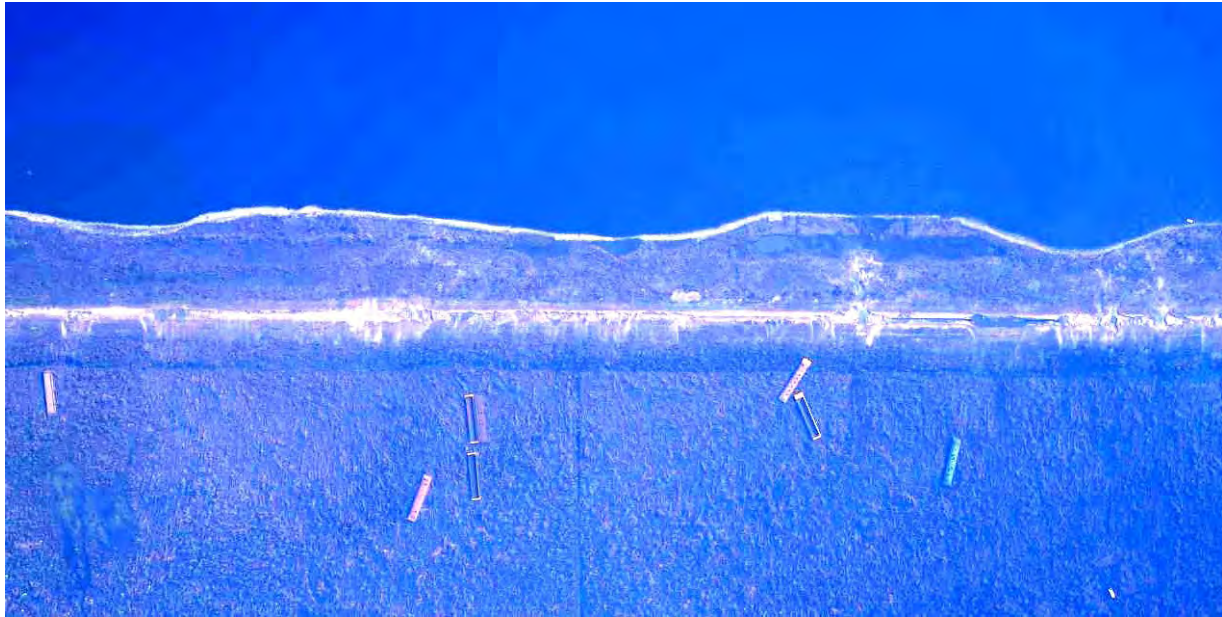


Figure 10.25: Scour pattern behind around the floodgate structure at the Bayou Bienvenue Control Structure shown in Figure 10.22.



Topographic map from Topozone.com

Figure 10.26: A U.S.G.S. topographic map showing the presence of “spoils” along the MRGO. As seen in Figure 10.28, the levee materials to the southeast of the Bayou Bienvenue Control Structure performed poorly.



Photograph courtesy ngs.woc.noaa.gov/Katrina/KATRINA0000.HTM

Figure 10.27: Aerial photograph taken by NOAA in early September 2005 along the MRGO showing severe erosion/breaches of the earthen levee and transport and deposition of large barges over the levee as a result of the storm surge (location 8 on Figure 10.6).



Photograph courtesy Les Harder

Figure 10.28: Close up aerial photograph of severely eroded MRGO levees at location 8 on Figure 10.6.



Photograph courtesy Les Harder

Figure 10.29: Control structure at Bayou Dupre (location 9 on Figure 10.6) suffered extensive scour and erosion during the storm surge and overtopping conditions associated with Hurricane Katrina.



Photograph courtesy Les Harder

Figure 10.30: Another aerial photograph of the scour and erosion damage to the Bayou Dupre control structure.



Figure 10.31: Areas of severe scour and erosion damage to the two control structures appear to correspond to the alignment of the abandoned original channel.



Photograph courtesy Robert Bea

Figure 10.32: Aerial photograph taken of the repair operations at Bayou Dupre in January 2006.



Photograph courtesy Robert Bea

Figure 10.33: Close up aerial photograph showing backfilling operations at Bayou Dupre.



Photograph courtesy Rune Storesund

Figure 10.34: Eastward looking view of the secondary earthen levee (constructed to approximately Elevation +6.0 feet MSL) immediately to the north of the Corinne Canal, east of Paris Road (location 10 on Figure 10.6). This levee was significantly overtopped and did not experience significant damage.



Photograph courtesy Rune Storesund

Figure 10.35: A fishing boat was washed over the levee shown in Figure 10.34 and landed in this Chalmette residential neighborhood within St. Bernard Parish.



Photograph courtesy Rune Storesund

Figure 10.36: The same levee shown in Figure 10.34, where a fishing boat washed over the levee during Hurricane Katrina, has now been raised from an elevation of approximately +6 feet MSL to an elevation of approximately +10 feet MSL by March of 2006.

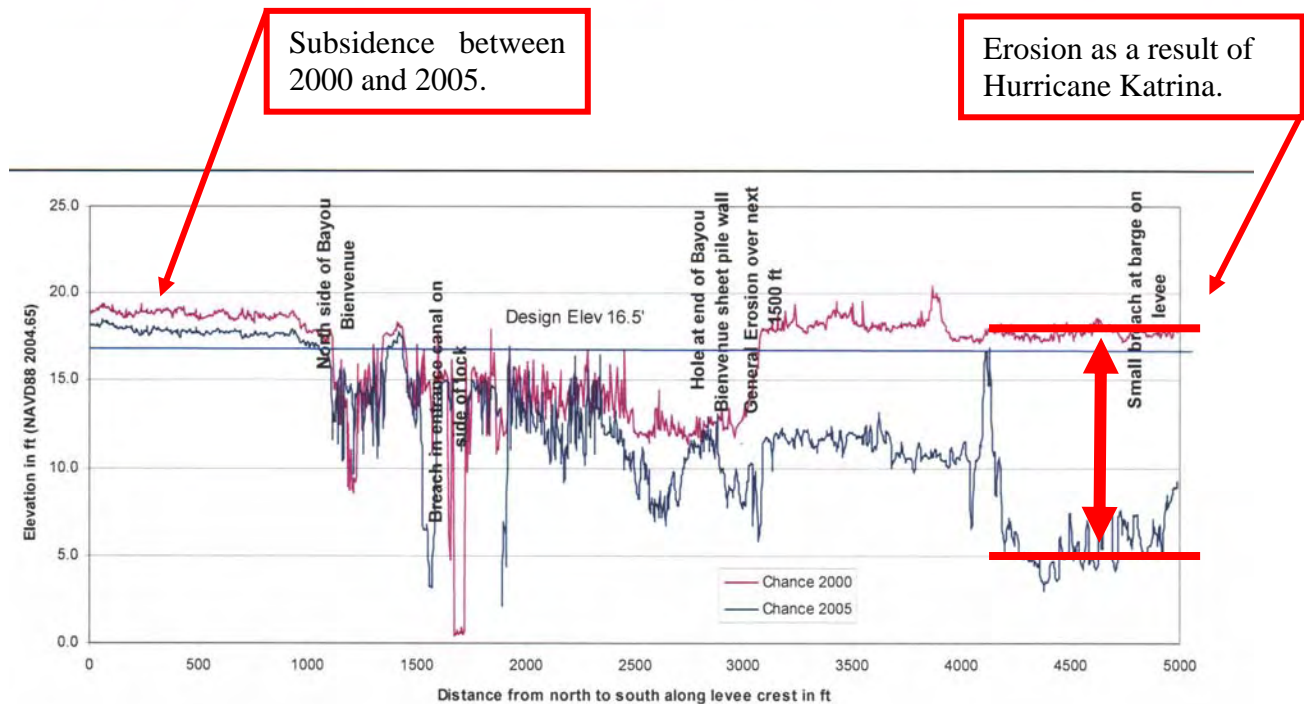
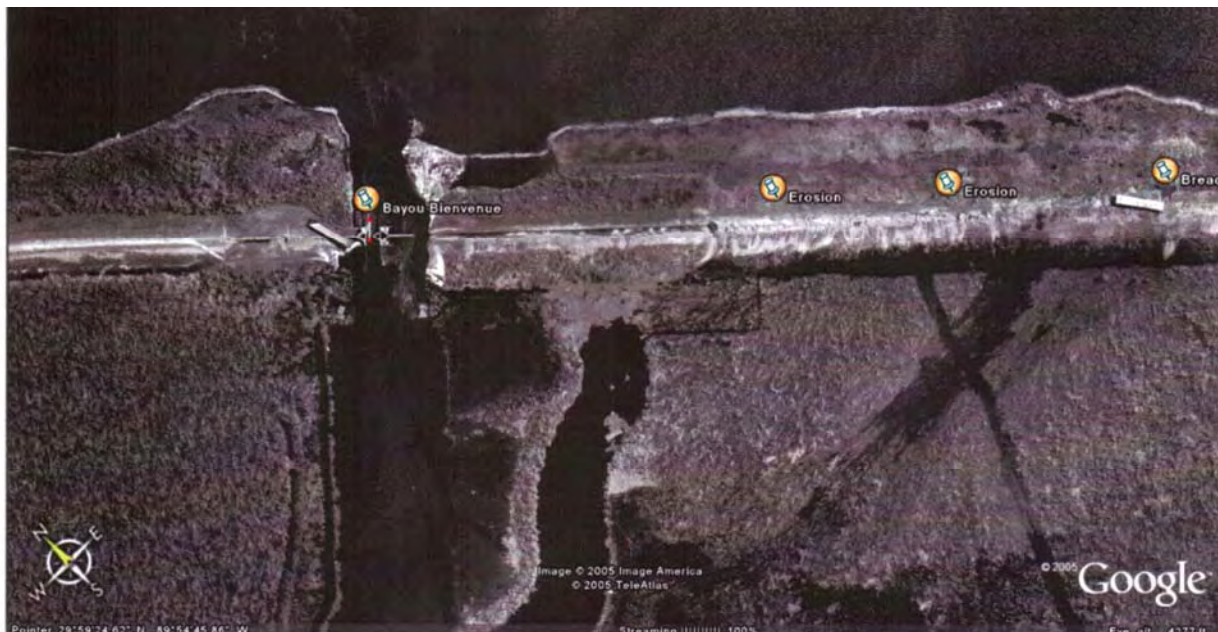


Figure 10.37: This is a comparison of two LiDAR surveys of the MRGO levee system at the Bayou Bienvenue control structure, near the intersection of the GIWW at the northeast corner of St. Bernard Parish. Effects of subsidence can be clearly seen in the elevation differences on the north side of the control structure and the control structure itself between 2000 and immediately following Hurricane Katrina in 2005. The levee on the north side of the Bayou Bienvenue control structure was largely undamaged. The levee on the south side of the control structure was catastrophically damaged. The magnitude of the erosion has been highlighted and white “splashes” of displaced levee materials can be seen in the aerial photograph (USACE 2006).



Figure 10.38: Levee sample sites within the Orleans East Protected Area.

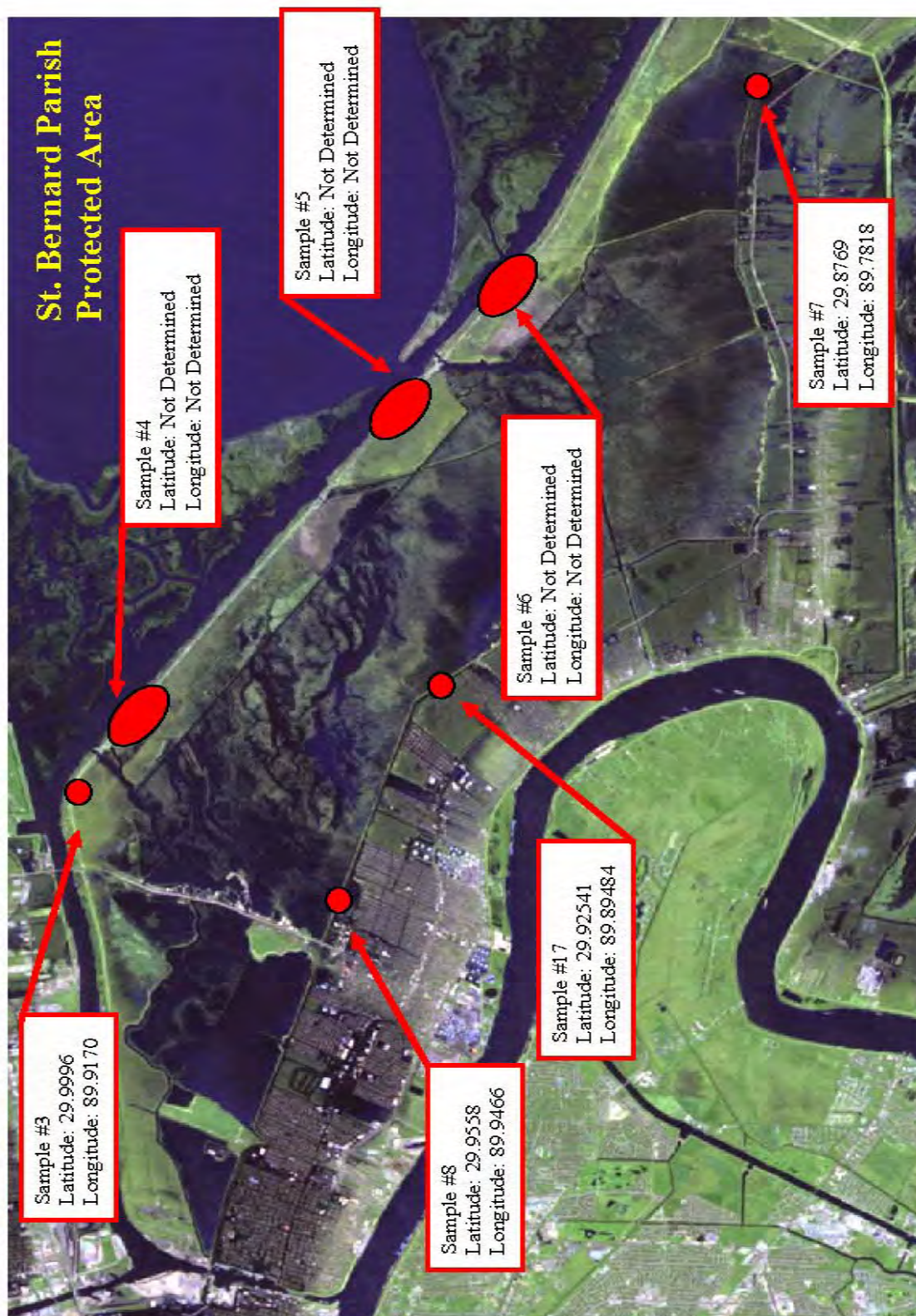


Figure 10.39: Levee sample sites within the St. Bernard Parish Protected Area.

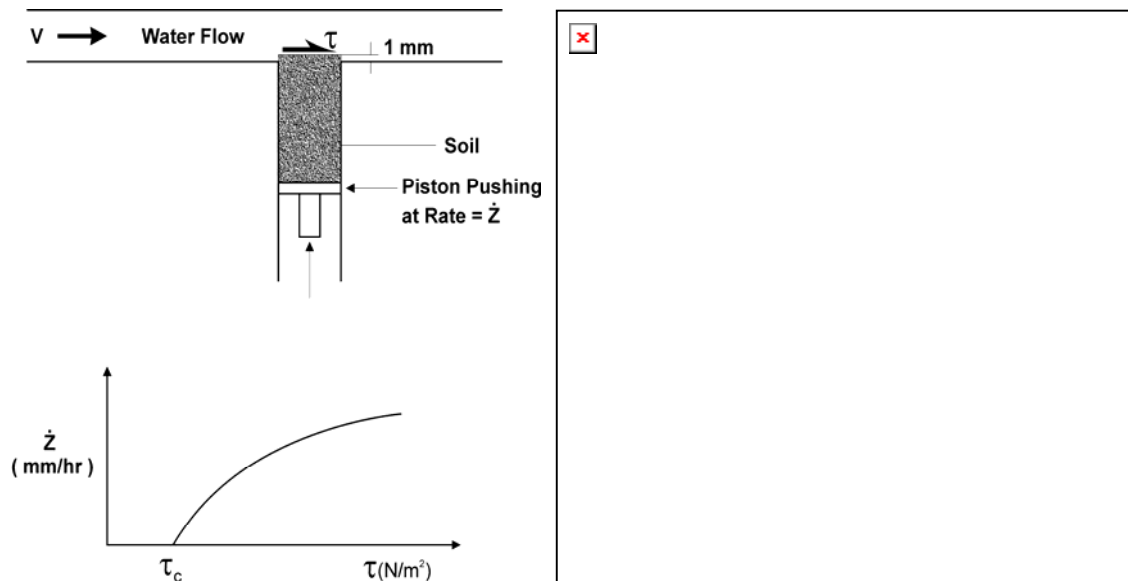


Figure 10.40: The EFA (Erosion Function Apparatus) as developed by Briaud et al. Soil samples are advanced into a rectangular tube of flowing water, creating a shear stress on top of the inserted soil sample. The velocity of the water is increased until the critical shear stress is achieved, where the soil begins to actively erode.

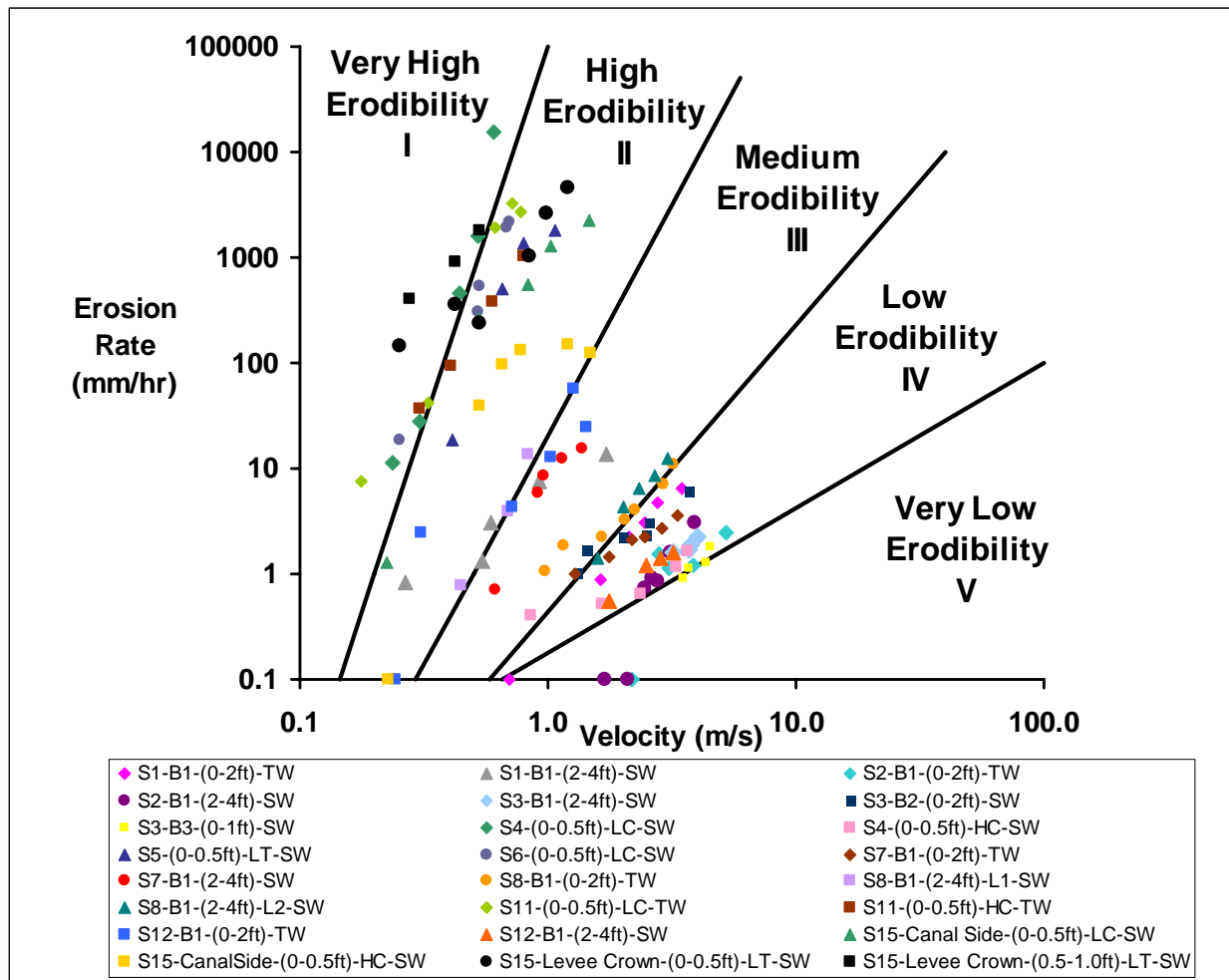
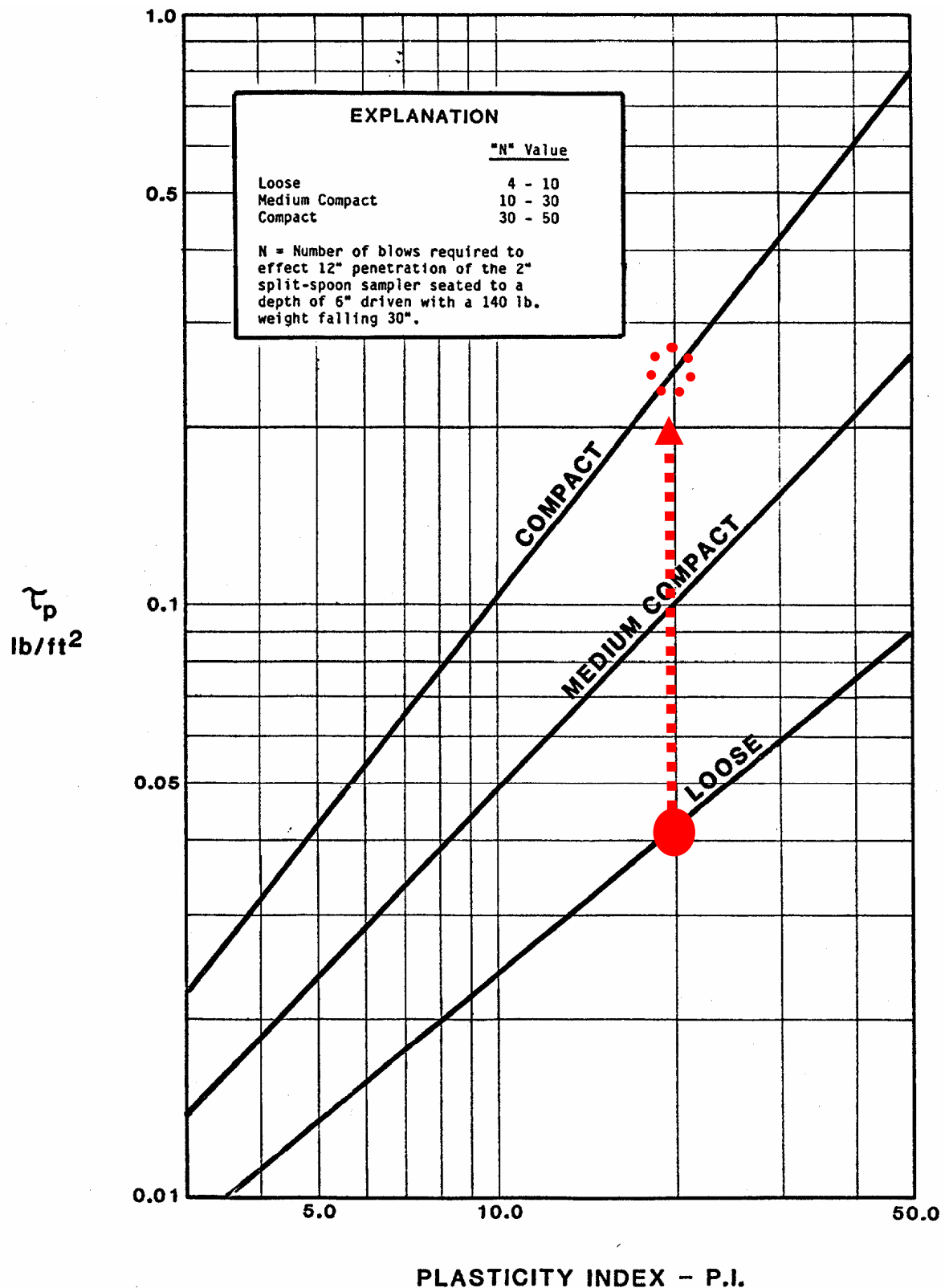


Figure 10.41: Summary of all EFA test results. The test result designations are based on the Site Number, the boring number at the site, the depth interval over which the sample was collected and sample notes. The EFA test results indicate that the materials used in levee construction varied from very high to very low erodibility, which matched the observed performance of these levees.



Graphic after FHWA 1988

Figure 10.42: The effects of compaction are clearly evident in this figure from this FHWA design guideline. For the same material (with a plasticity index of 20) a ten-fold increase in shear stress capacity can be achieved by properly compacting the material.

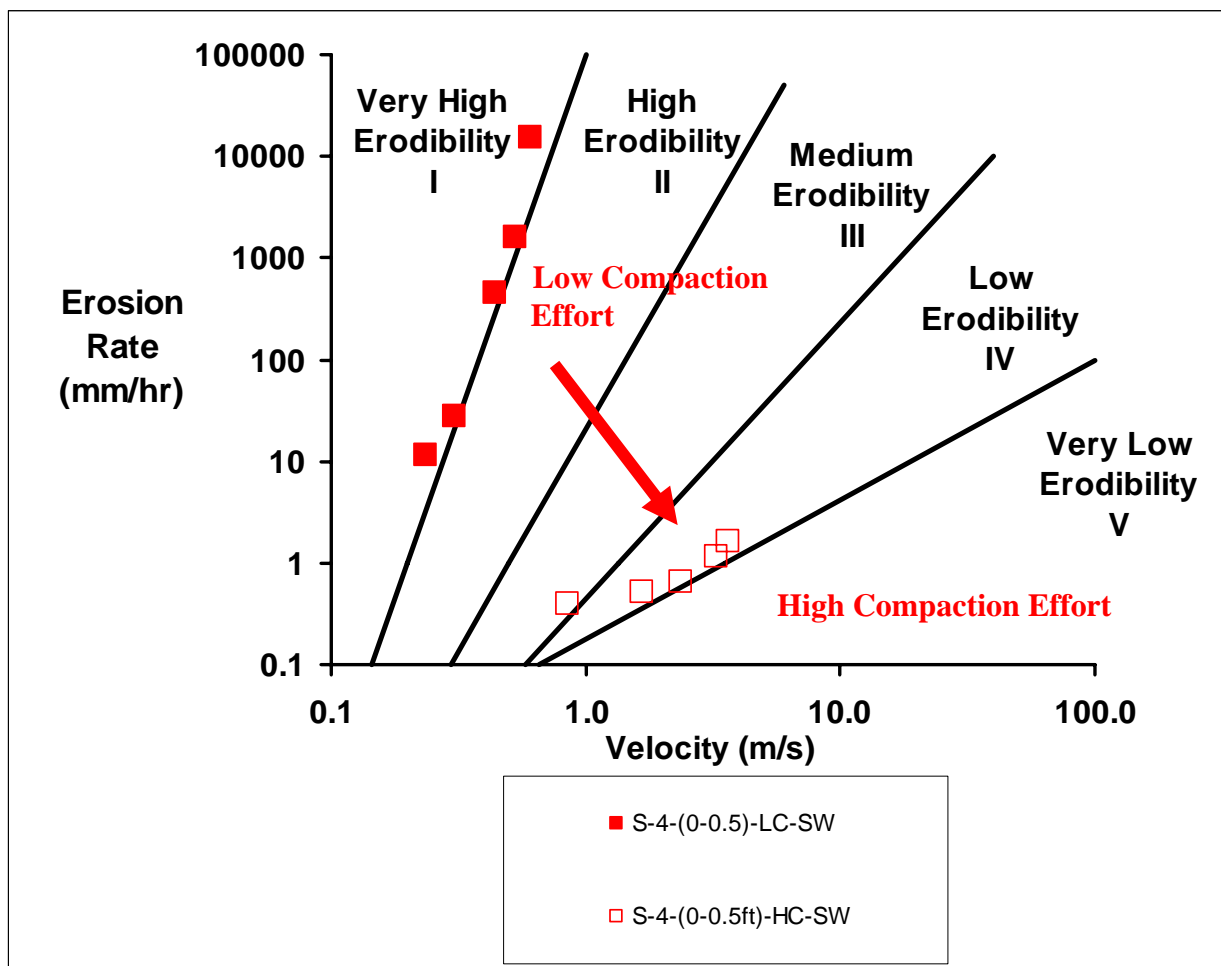


Figure 10.43: Here materials sampled from the MRGO were tested at two levels of compaction. This figure shows the dramatic impact proper compaction has on the erodibility of soils. Materials sampled from the MRGO levee were tested at two compaction levels: low compactive effort and high compaction effort. The corresponding results speak volumes to the importance of compaction in earthen levees. The low-compaction sample was found to be very highly erodible, whereas the high-compaction sample exhibited low erodibility characteristics.

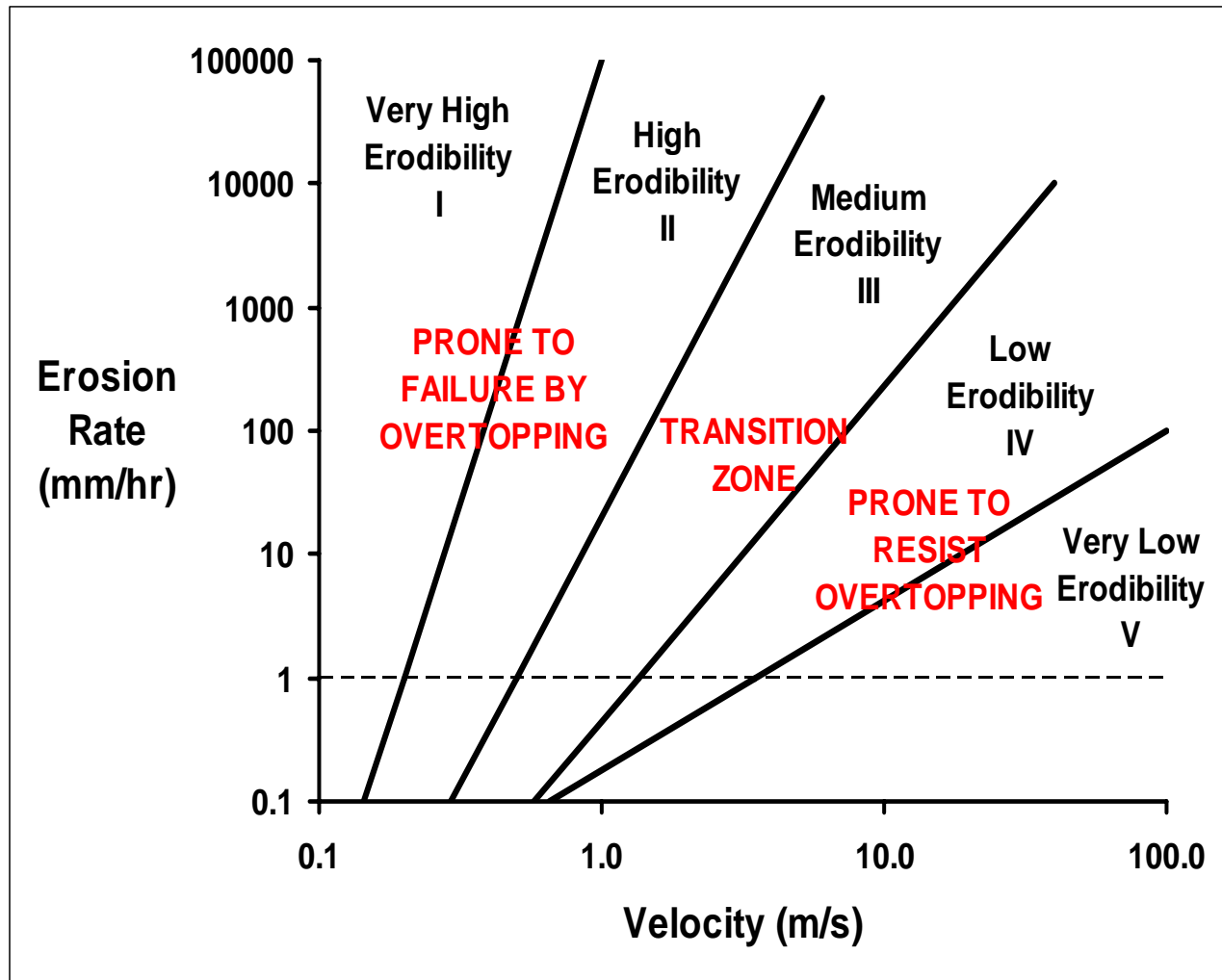


Figure 10.44: Resulting guideline table for evaluating erosion susceptibility of soils used for levee construction developed by Dr. Briaud from Texas A & M University.

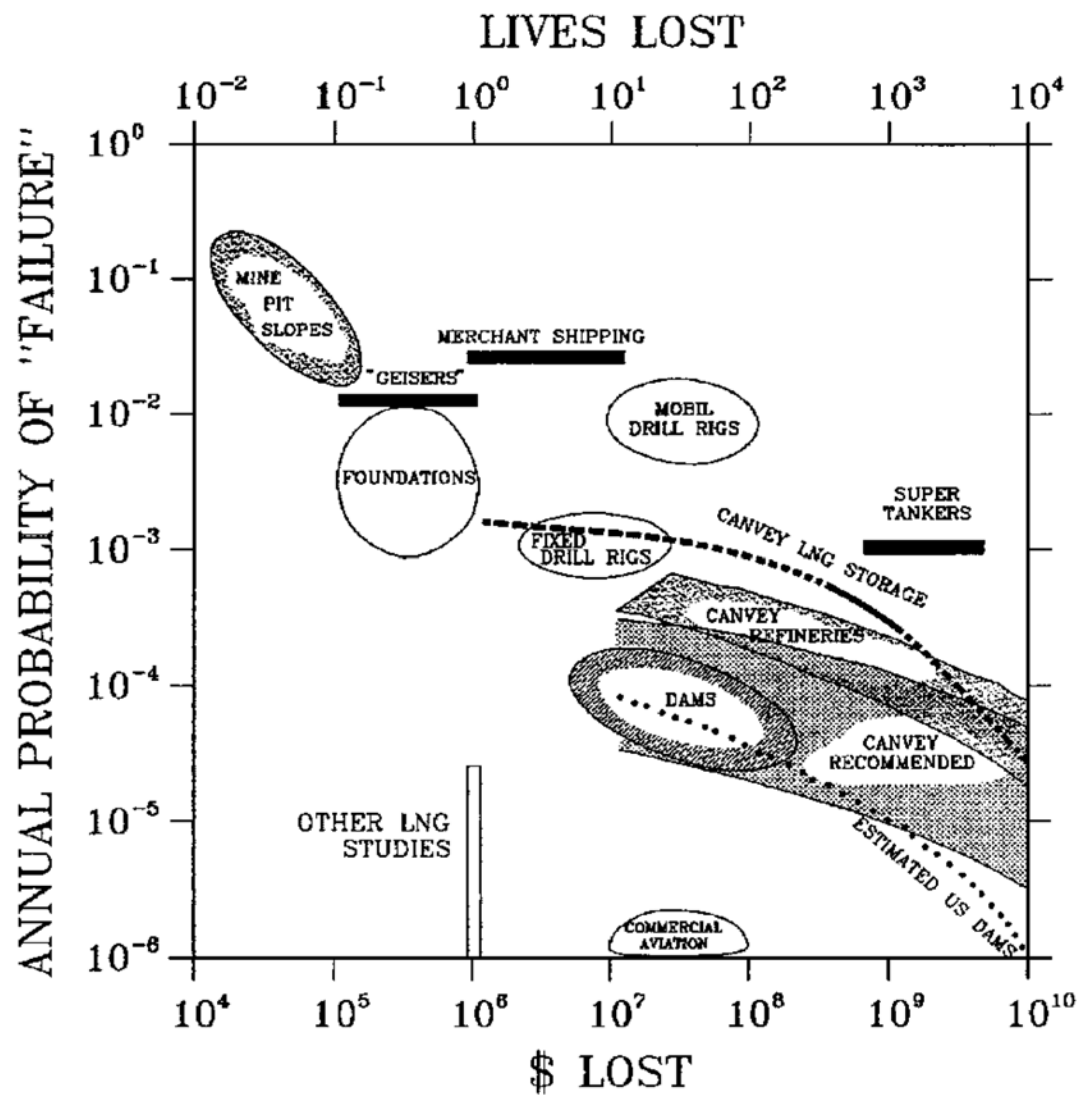


Figure 10.45: Plot of back-calculated annual probability of failure vs. lives lost and \$ lost (note that either the "lives lost" or "\$ lost" axes are used, they are not intended to be used in conjunction). From Christian, 2004.

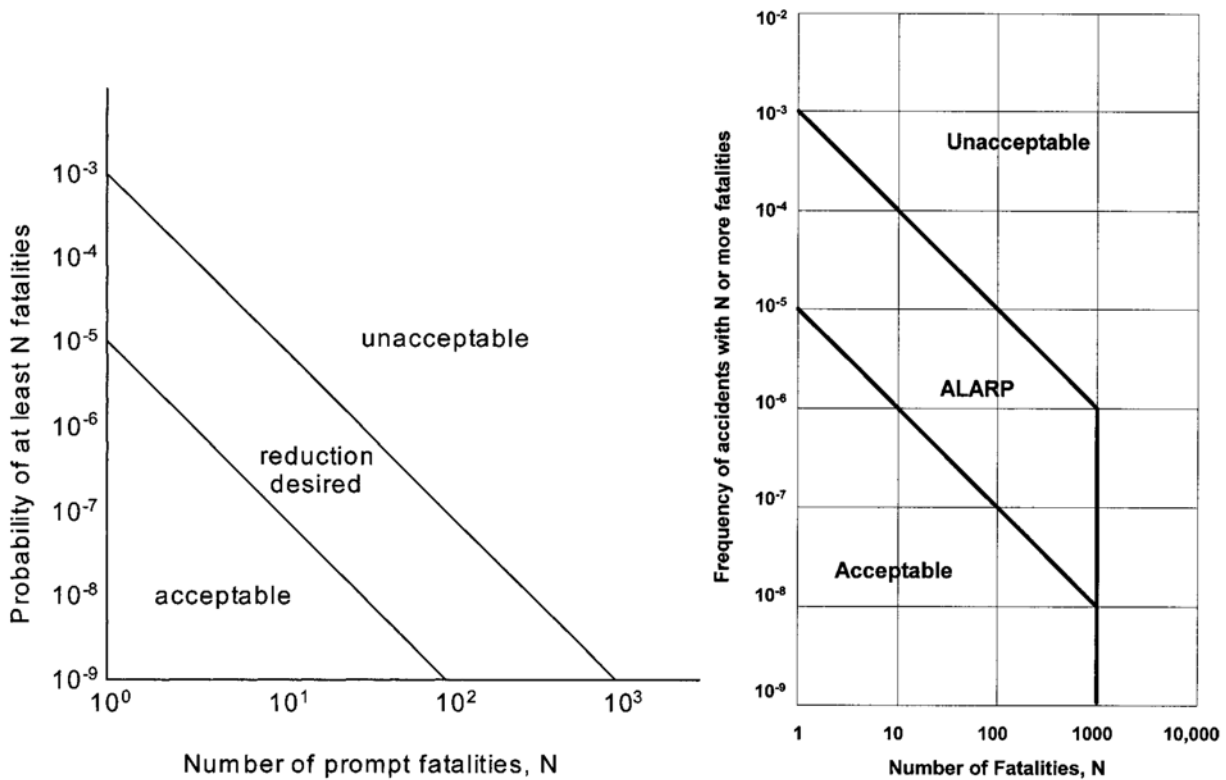


Figure 10.46: F-N diagrams adopted by the Hong Kong Planning Department (left) and F-N diagram as proposed for planning and design use in the Netherlands (right). From Christian, 2004.

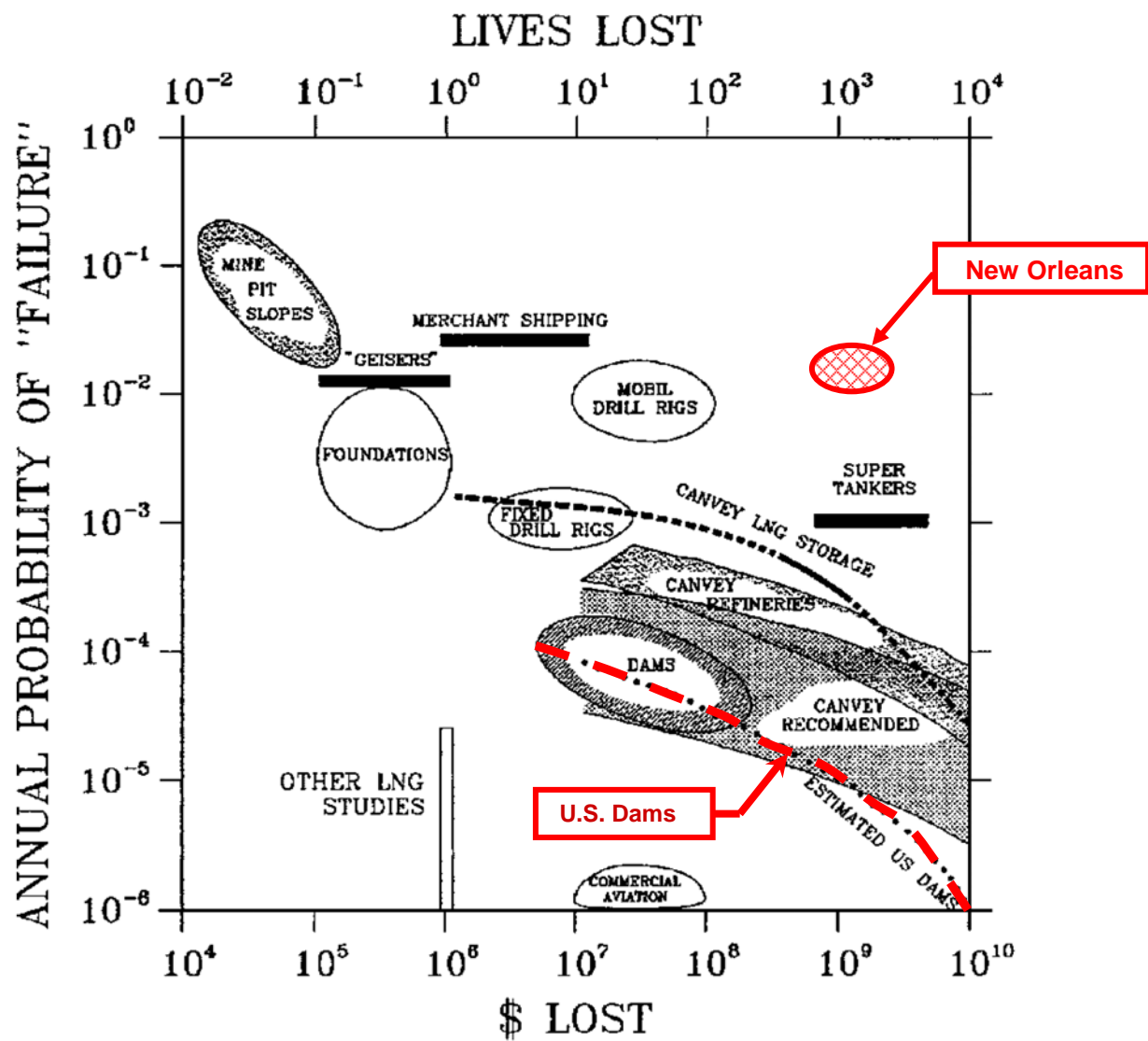


Figure 10.47: Figure 10.45 repeated, with approximate level of reliability and consequences for the New Orleans regional flood protection systems indicated, and with current U.S. practice for dams highlighted (red dashed line.)

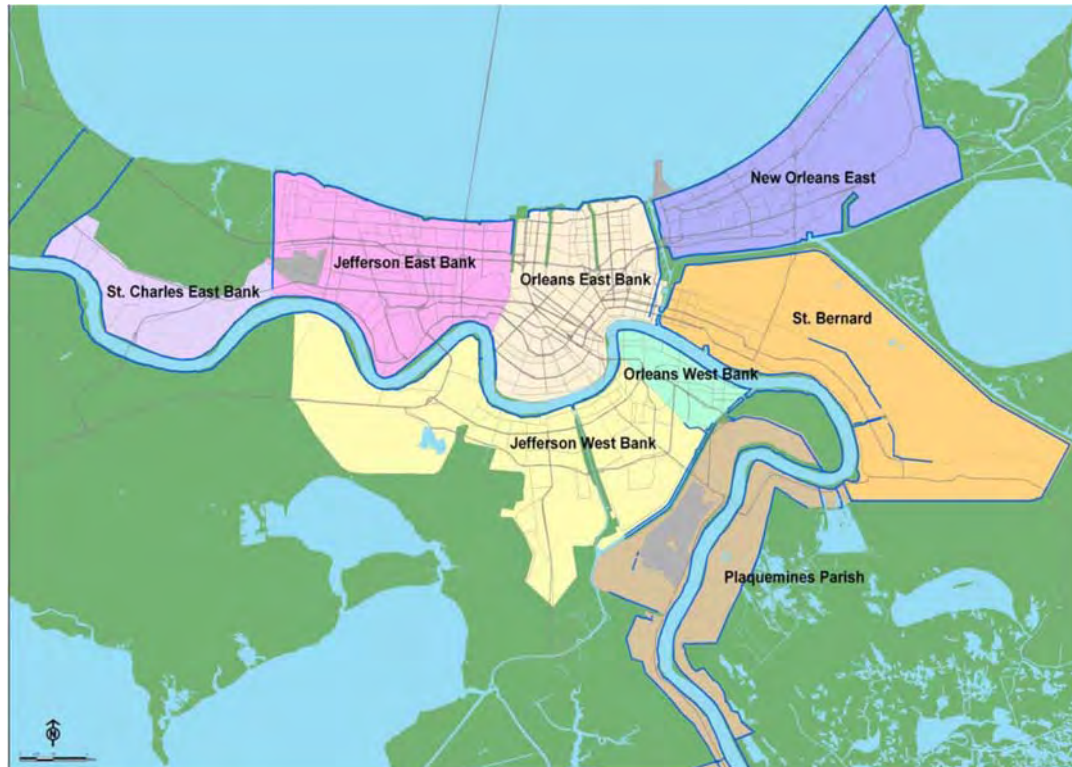


Figure 10.48: Overview of the drainage and pumping parishes studied as part of the IPET (2006) performance evaluation. The primary study areas were Jefferson, Orleans, Plaquemines, and St. Bernard parishes. [IPET; June 1, 2006]

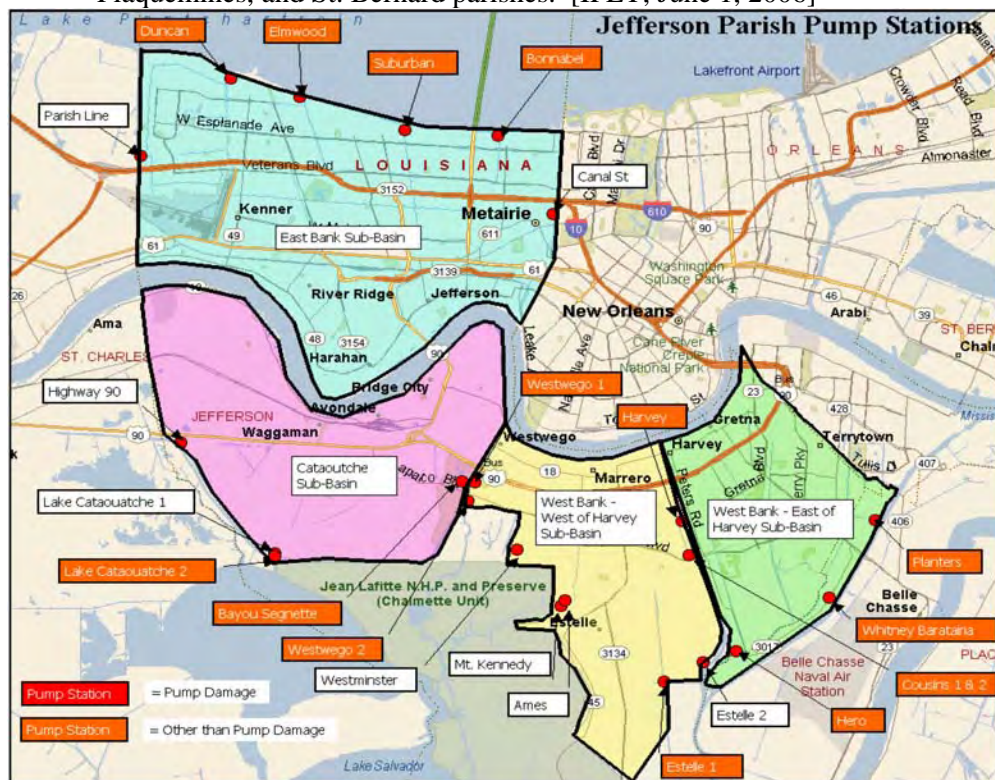


Figure 10.49: Detailed map of pump stations within Jefferson parish.
[IPET; June 1, 2006]

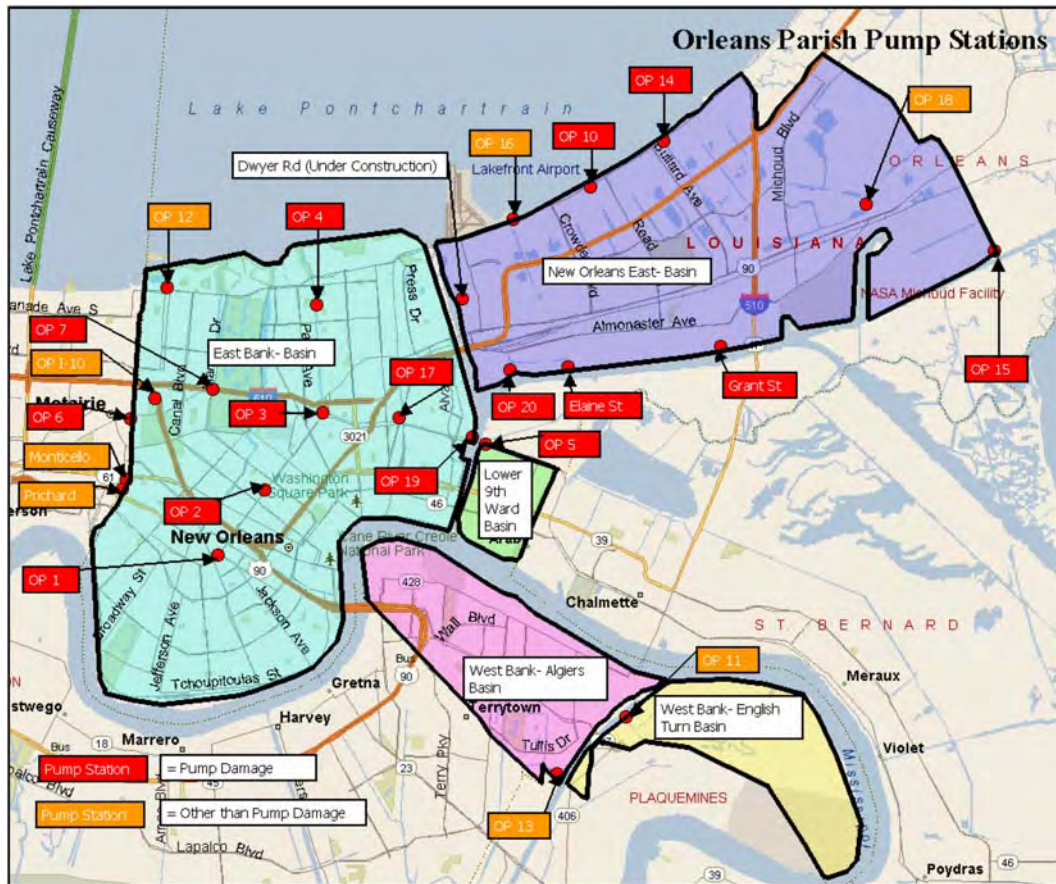


Figure 10.50: Detailed map of pump stations within Orleans parish. [IPET; June 1,2006]

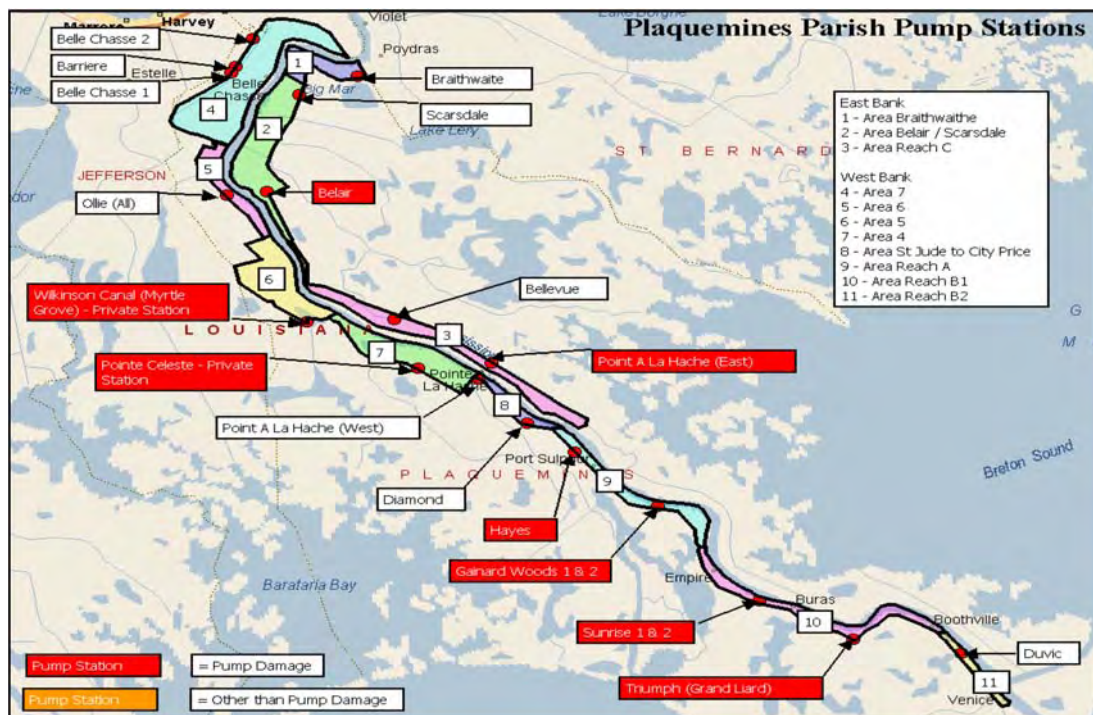


Figure 10.51: Detailed map of pump stations within Plaquemines parish. [IPET; June 1, 2006]

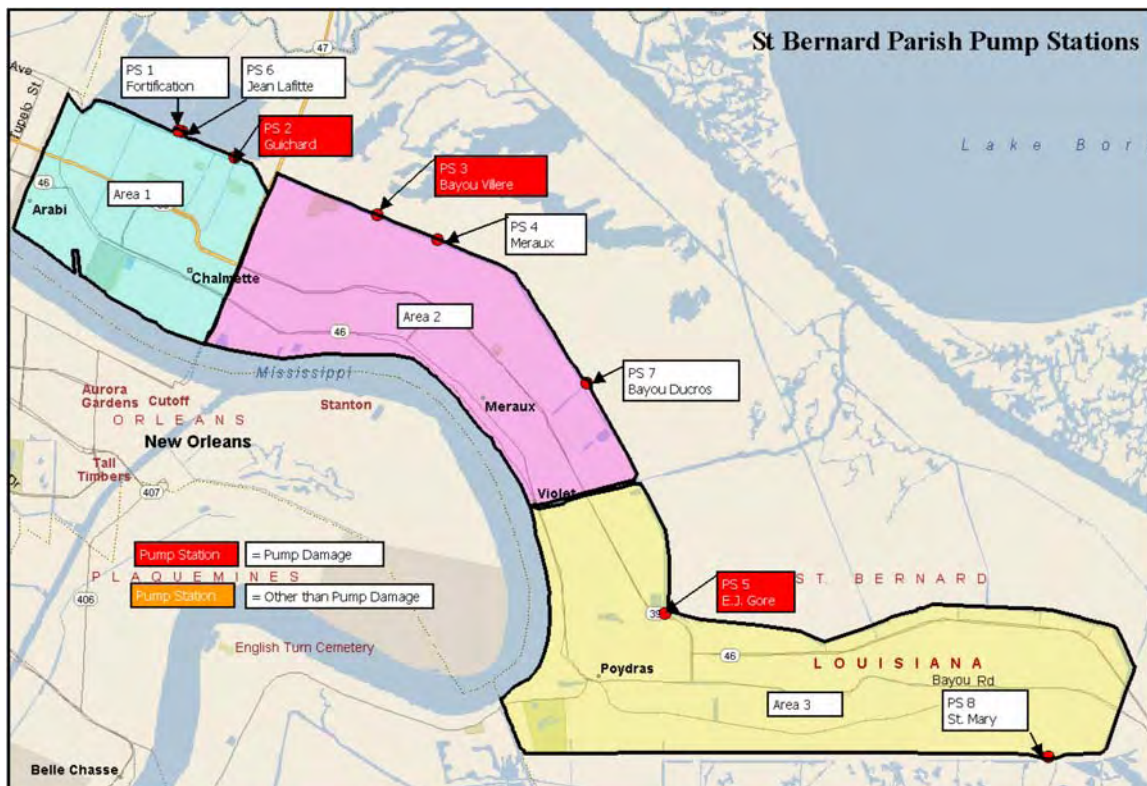
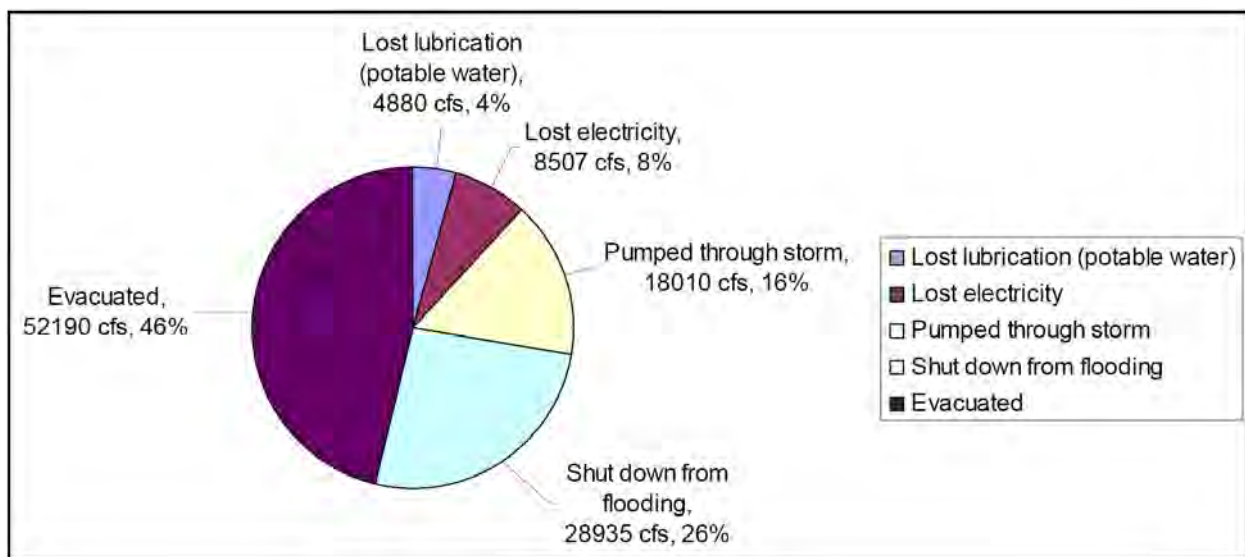


Figure 10.52: Detailed map of pump stations within St. Bernard parish. [IPET, June 1, 2006]



Source: IPET, 2006

Figure 10.53: Performance of the pumping system (and causes of pumping capacity loss) during and after hurricane Katrina.

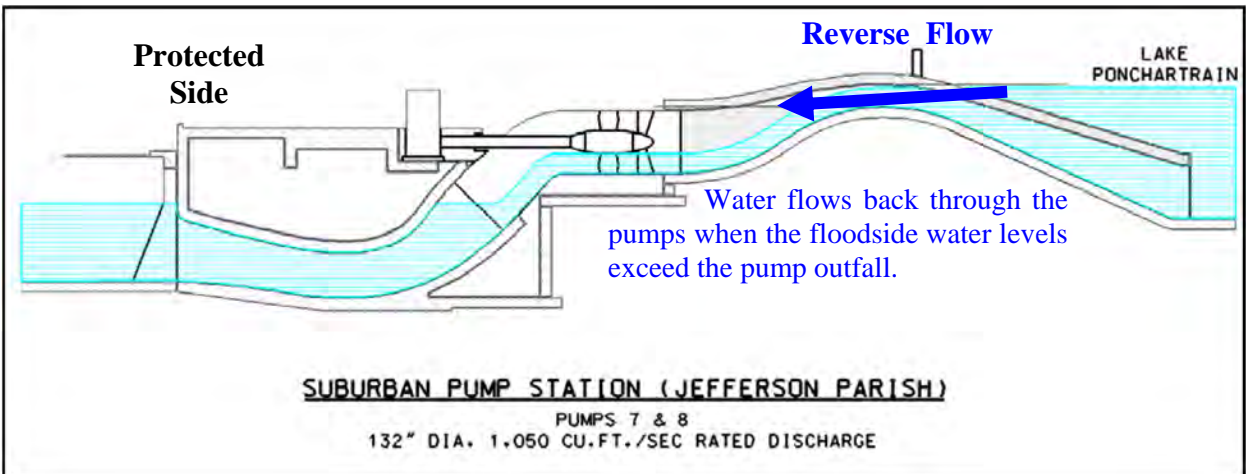


Figure 10.54: Conditions where the water level on the outboard side of the pumping station is higher than the pump infrastructure, resulting in reverse flow from the flood side back into the “dry” side. [IPET; June 1, 2006]

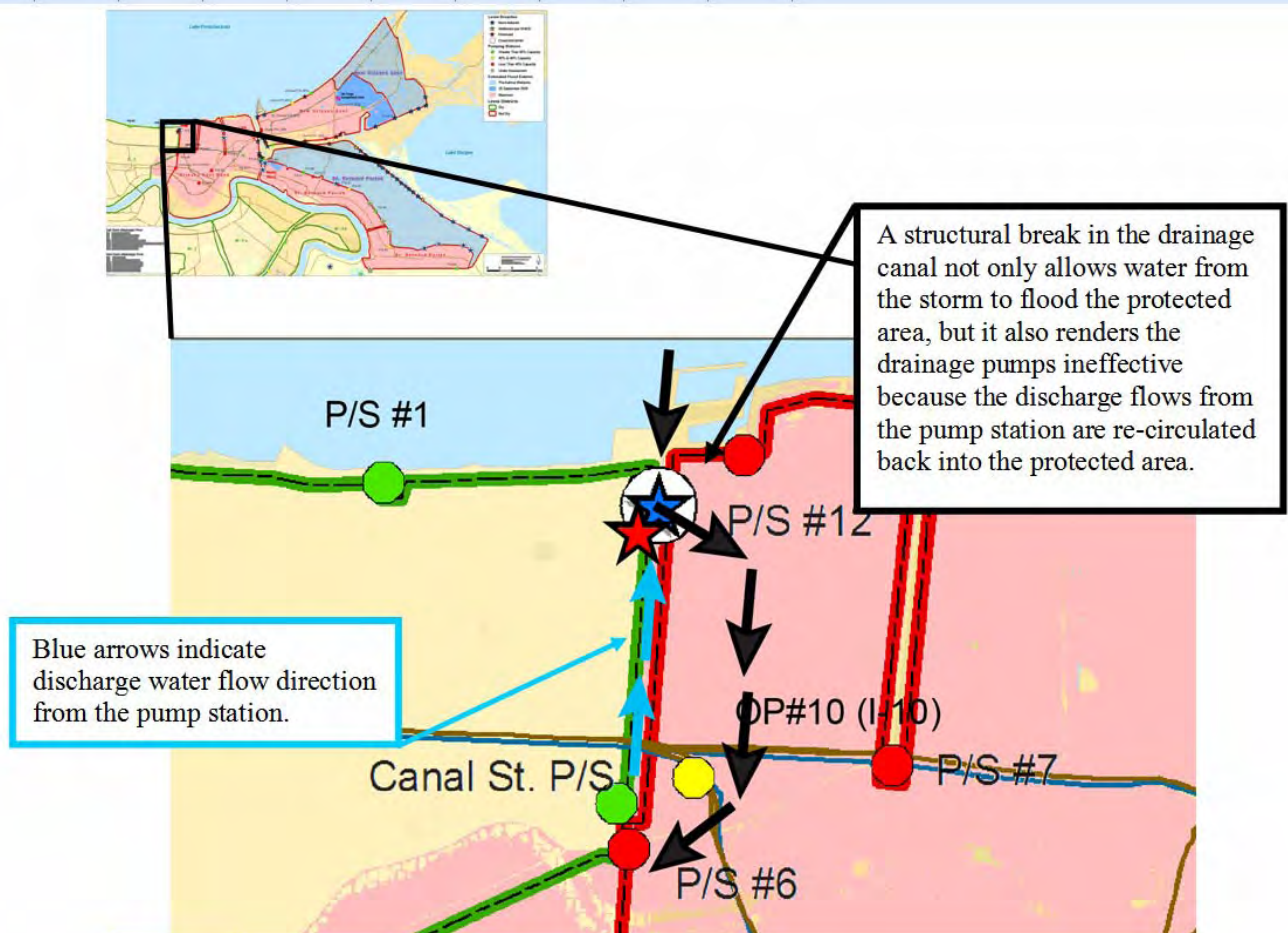


Figure 10.55: Breakdown of the pumping and drainage system as a result of structural failures within the drainage canals. Flood waters and discharge water from the pumping station flow back into the protected area where structural failures have occurred.

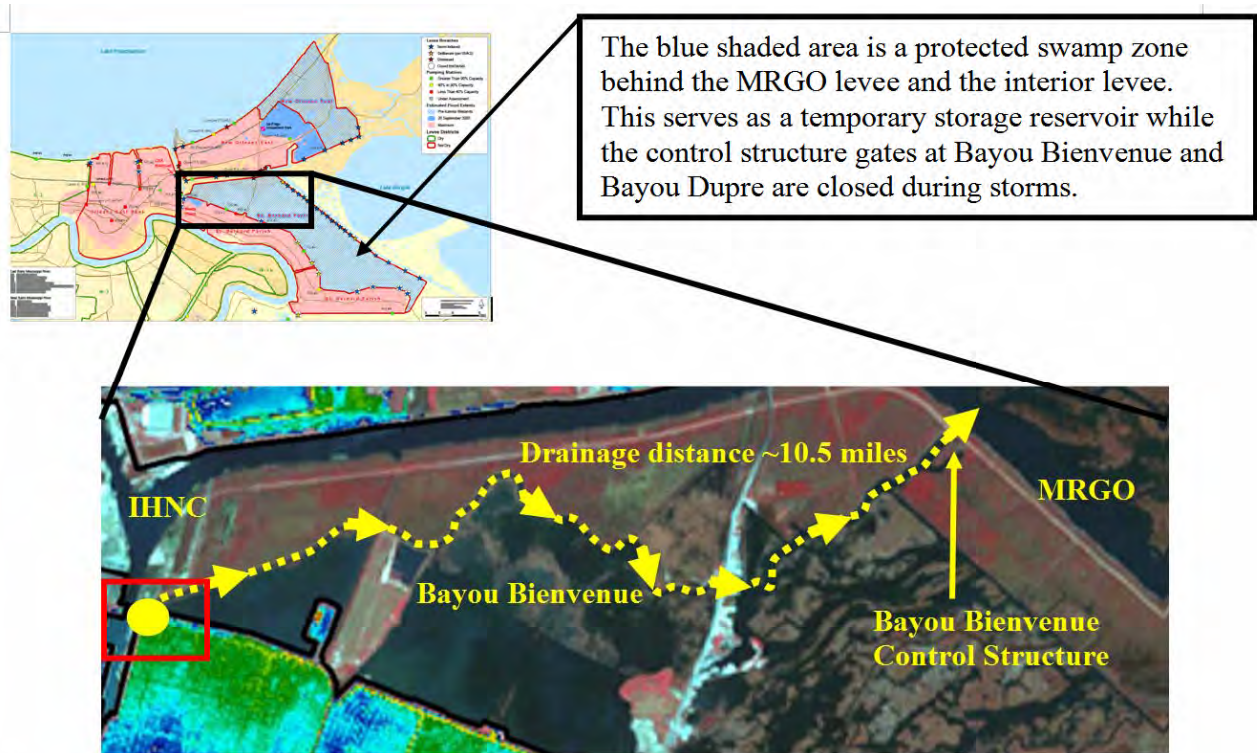


Figure 10.56(a): The current drainage configuration for the lower 9th Ward in St. Bernard parish starts at the Florida Street pump stations and flows approximately 10.5 miles through Bayou Bienvenue, then through a control structure to the MRGO.

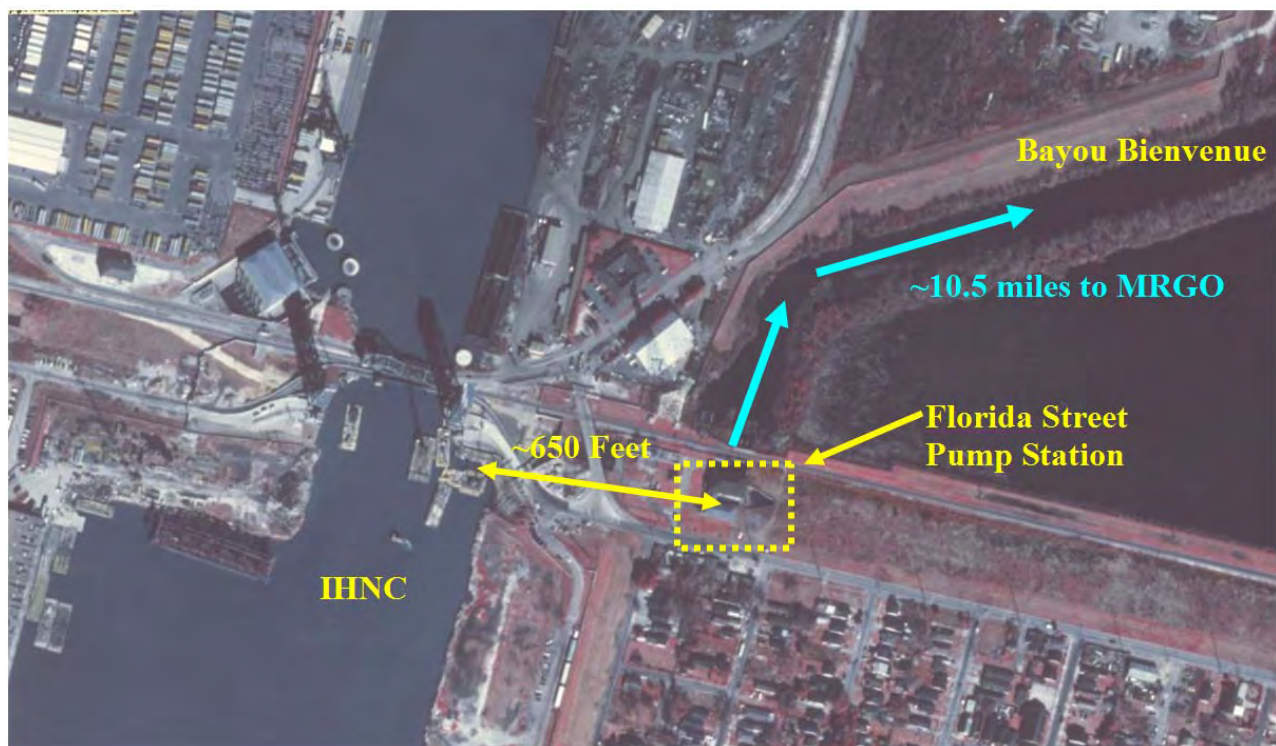


Figure 10.56(b): The alternative; pumping into the IHNC 650 feet to the west.

CHAPTER ELEVEN: SUMMARY OF ENGINEERING LESSONS

11.1 Introduction

This chapter presents a summary overview of the principal technical lessons and findings from this investigation. The next three chapters that follow then carry forward a study of the underlying organizational, institutional, political, economic, human factors and decision-making issues that arise in conjunction with these “engineering” lessons and findings.

11.2 Overarching Strategic Issues

11.2.1 Targeted Levels of Safety and Reliability

Figure 11.1 shows a “risk plot” with the vertical axis representing the annual likelihood of failure, and the horizontal axis representing the expected cost of such failure either in dollars (bottom axis) or in lives lost (top axis). This figure shows the ranges of risk, or reliability, representing common practice for a number of areas of human endeavor. Highlighted with a heavy red dashed line near the bottom is current U. S. practice in the field of dam engineering.

Also shown on this plot is our investigation’s view of the level of reliability associated with the New Orleans regional flood protection systems prior to hurricane Katrina. Our best estimate, based on information currently available, and given the targeted design levels and the flaws and vulnerabilities embedded in the system, is that the pre-Katrina system was likely to fail catastrophically approximately every 30 to 75 years. The cost of the failure (in hurricane Katrina) was on the order of \$100 to \$200 billion in losses, and approximately 1,500 lives were lost.

There is a stark contrast between the levels of reliability for which major U.S. dams are engineered, and the level of reliability of the New Orleans levee systems. This is true, to only slightly varying degree, for most levee and flood protection systems across the entire nation.

“Dams” are engineered to very high levels of reliability because their potential failure would threaten large numbers of lives, and large economic losses as well. Few dams protect (or threaten) populations as large as the combined greater New Orleans and adjoining Jefferson parish region, however, and simple logic would suggest that flood protection systems defending large populations like this should be targeted at similar levels of safety or reliability.

As indicated by the large arrow in Figure 11.1, the difference between the level of risk (or reliability) of the New Orleans regional flood protection systems and conventional U.S. dam practice is approximately three orders of magnitude; a factor of roughly 1,000 times safer and more reliable. Reliability is a function of two sub-elements: (1) the targeted level of

loading(s) to be handled, and (2) the reliability with which that target is met. Achieving significantly higher levels of reliability for complex regional flood protection systems like that of the New Orleans region would require major improvements in both sub-elements.

The New Orleans regional flood protection systems were never specifically targeted at any given level of reliability with regard to formal definition of storm levels for design (e.g.: the Standard Project Hurricane was never quantified as a “100-year storm” or a “300-year storm”, etc.), and this was a lapse, as it put the design of the regional flood protection system out of step with current practice. Our assessment in plotting the pre-Katrina New Orleans case in Figure 11.1 is that the system as it existed was likely to be failed by roughly the 30-year to the 75-year (average recurrence interval) storm.

The actual design intent was for more, though how much more was never formally defined.

In addition, the system did not perform as intended; multiple failures occurred at levels of storm surge and wave loading that were less than or equal to what many of the failed system elements had been intended to safely handle.

If the system is re-engineered to safely (successfully) handle a 100-year storm (storm loading likely to be exceeded typically once every 100 years), then the large red area of Figure 11.1 would move to the location shown by the light blue area in Figure 11.2. That would not bring the level of safety and reliability anywhere near to current U.S. practice for dams. Targeting a 1,000-year level of flood protection, and achieving that level, would result in the darker blue zone in Figure 11.2.

There are some significant challenges involved at the decision-making and policy levels regarding appropriate levels of safety (e.g. storm levels) for which such systems should be designed, and the degree to which resources should be committed to achieve this.

The other element of risk/reliability is the degree to which targeted design levels are successfully achieved. Levels of success were not good in hurricane Katrina; numerous failures occurred at storm surge and wave levels less than or equal to those for which the flood protection system elements were intended to be designed. These failures occurred because margins for error (e.g.: design Factors of Safety) were inappropriately small, because decisions were made to reduce costs in exchange for increased levels of risk, and because of errors and lapses in design, construction and maintenance. These types of “engineering” issues are discussed in Section 11.3 that follows.

11.2.2 Funding and Appropriations

A second set of overarching considerations are those associated with the Byzantine process by which large, complex, regional-scale flood protection systems are conceived, approved, designed, funded, constructed, maintained and operated. No useful discussion of engineering challenges can proceed without noting the tremendous additional difficulties that arise due to these types of “non-engineering” issues.

Figure 11.3 presents a very simplified schematic illustration of the principal “technical” steps involved in creating and operating a regional flood protection system (left side of the figure, in blue), and also the corollary political, organizational, institutional, engineering and construction units and entities (right side of the figure, in green) that must interact to foment this process. This is discussed in detail in Chapters 12 through 14, so we will simply note here that it is not realistic to assume that we can achieve the significant improvement of the safety and reliability of the New Orleans regional flood protection system that appears warranted simply by making adjustments to the “technical” side of this figure. Simply revising design manuals and engineering procedures, etc., cannot possibly achieve the significant improvement in system reliability that should be sought; significant improvements on the right-hand side of this figure will be needed as well.

These issues are addressed in Chapters 12 through 14, but several key issues warrant special mention at this stage. The first of these is funding and appropriations; the allocation of resources to the creation and operation of the regional flood protection system. This allocation of resources is, properly, the domain of the decision-making bodies involved; elected representatives (government) at both the federal and local levels. Unfortunately, these elected officials often lose track of the ramifications of their decisions with regard to complex technical systems created and operated over long periods of time.

It should not take 50 years to construct a critical system providing life-safety for a region with a population of nearly one million people (the greater New Orleans/Jefferson parish region.) The regional flood protection system was incomplete at the time of Katrina’s arrival; it was intended to be complete by the year 2015, fully 50 years after its inception in response to the catastrophic flooding of New Orleans produced by hurricane Betsy in 1965. We need to do better.

Apart from the obvious need to more rapidly and effectively provide protection for large numbers of citizens, these types of extended construction periods (covering multiple decades) wreak havoc with the actual engineering and construction of the intended systems themselves. As noted in the IPET Draft Final Report (IPET; June 1, 2006), the New Orleans regional flood protection system was largely a system in name only. Having been constructed over four-plus decades, and in innumerable individual segments and sections, it was optimistic to expect that the various inter-connecting elements would function perfectly well together. Stretching the construction over multiple decades posed major challenges with regard to progressive loss of institutional memory and expertise, and it required excessive segmentation of systems that needed to function literally seamlessly as contiguous defenses.

Another difficult issue was the nearly constant pressure to reduce costs. Decisions that produced reductions in the costs of specific flood protection elements routinely resulted in corollary increases in levels of risk; the increased likelihood that the system would not perform well when eventually tested. As discussed in Section 11.3, this type of trade-off between short-term cost reductions and increased risk now appears very hard to justify, as it contributed significantly to many of the specific failures and breaches during hurricane Katrina, and resulted in catastrophic losses that now dwarf the short-term savings by two orders of magnitude and more.

11.3 Principal Engineering Findings and Lessons

11.3.1 Introduction and Overview

Figure 11.4 shows an overview of the New Orleans area, indicating the locations of the principal failures, breaches, and distressed sections of the New Orleans regional flood protection system studied in this investigation. Plaquemines parish (along the lower reach of the Mississippi River) is not included in this figure; instead it is indicated by the large arrow at the bottom. The individual features, and groups of features, in Figure 11.4 are numbered for purposes of discussion. Table 11.1 presents a summary of issues at each of these locations, using the same numbering scheme as Figure 11.4.

The New Orleans regional flood protection system failed massively and catastrophically during hurricane Katrina. Depending on how one counts individual breaches (or groups breaches extending along long frontages that were massively eroded and scoured), the number of failed sections was somewhere between three dozen to 50-plus.

For an overview of performance, the system can be roughly sub-divided into four zones. At the southern end, the flood protection systems in Plaquemines Parish were massively overwhelmed by storm surges and waves significantly more severe than they had been designed to handle.

At the east flank (fronting Lake Borgne), and in the central region (along the IHNC and the GIWW/MRGO channels) the storm loadings were approximately equal to those for which the system was intended to be designed. Design loading conditions were exceeded at some locations, especially along the Lake Borgne frontages, but intended design levels were only slightly exceed and the system might have been expected to perform better. Instead, massive and catastrophic breaches occurred at multiple locations. These were principally the result of one or more of the following: (1) insufficient crest heights (which led to, and exacerbated, overtopping problems), (2) use of inappropriate materials at some locations (materials with very poor resistance to erosion), and (3) other engineering lapses and oversights.

At the north end, along the Lake Pontchartrain frontage, storm surge levels and waves presented lesser levels of loading than the system elements were intended to safely handle. System performance was good along most of the lake frontage itself, but three catastrophic breaches occurred along the drainage canals at the north end of the main (downtown) New Orleans protected basin, and these were the principal source of approximately 85% of the floodwaters that catastrophically inundated most of that basin. These three failures, together, accounted for nearly half of the overall loss of life in this event, and a similar fraction of the overall economic losses and property damages. These three major failures on the drainage canals were the result of engineering failures in design.

Multiple issues and challenges contributed to the numerous individual failures, and these have been discussed in Chapters 3 through 10. The discussion that follows will select highlights with regard to lessons that can be extracted from this event with an eye towards effecting better system performance in the future.

11.3.2 Plaquemines Parish

Plaquemines parish is a narrow, highly exposed, and sparsely populated set of corridors along the edges of the lower reaches of the Mississippi River, extending south from New Orleans to the river's outlet in the Gulf. This protected strip, with "river" levees fronting the Mississippi River and a second, parallel set of "storm" levees facing away from the river forming a protected corridor less than a mile wide, serves to protect a number of small communities as well as utilities and pipelines. This protected corridor also provides protected access for workers and supplies servicing the large offshore oil fields out in the Gulf of Mexico.

The flood protection systems of lower Plaquemines parish were massively overtopped and overwhelmed by storm surge and storm waves that significantly exceeded design levels, and multiple breaches and failures resulted (see Chapter 5.)

Plaquemines parish is sparsely populated, with a pre-Katrina population of only about 27,000 people. Given increased public awareness of risk and exposure, less than half of these are expected to return. There are few engineering lessons to be learned from the experience of Plaquemines parish; when even well-constructed levees and floodwalls are sufficiently massively overwhelmed, failures will occur. The lesson, if anything, is one of humility in the face of nature. If there is an engineering lesson here, it is:

- 1. Not all areas can be protected, and it is not economically reasonable to commit out of scale resources to the protection of some areas. We must learn to choose our battles.*

Federal policy across the nation over the past two decades has been moving increasingly away from out of scale expenditures to protect, or to insure, small populations living on marginal lands at high risk with respect to flooding. Plaquemines parish will represent an interesting case in this regard.

11.3.3 The East Flank; New Orleans East and the St. Bernard/Lower Ninth Ward Protected Areas

Major cities are different. A de facto decision has already been made to reconstruct New Orleans, and to upgrade its regional flood protection systems. Accordingly, it is now incumbent upon us to do all that we can to extract important engineering lessons from the Katrina experience, and to see that these lessons are suitably applied to efforts to improve the levels of safety and reliability of the regional flood protection systems in future events.

The main breaches that were the principal source of flooding for both the St. Bernard/Lower Ninth Ward protected area and the New Orleans East protected area were the levee frontages facing "Lake" Borgne (which is actually a bay, as it is connected directly to the Gulf of Mexico.) These are sections #2 (and 2a) and #3 in Figure 11.4, and in Table 11.1.

These two sections shared a number of fatal characteristics. Both sections were constructed largely using materials dredged from the excavations for the adjacent shipping

channels (the MRGO channel and the GIWW channel, respectively), and as a result both levee frontages including large sections of levees comprised in large part of materials known to perform very poorly with regard to erosion. These unacceptably erodeable materials included sands and lightweight shell-sands, and the massive and catastrophic erosion of these materials caused the rapid failure of great lengths of levees along both the MRGO and GIWW frontages. Another commonality was the lack of swamps or cypress groves on the outboard sides of these levee frontages; features that would have served to dampen (reduce the energy and intensity) of storm-driven waves attacking these frontages. Finally, these two frontages were also, unfortunately, the two frontages that were most directly exposed to severe wind-driven waves across a large body of open water.

As a result, these two frontages failed catastrophically, and were massively eroded along multiple miles of frontage, creating long breaches through which the hurricane storm surge passed easily, and with devastating consequences for the communities of these two protected areas.

Interestingly, adjacent levee sections along these same frontages, although also overtopped, performed well; suffering relatively minor erosion and continuing to provide protection as the storm surge subsided after the period of overtopping during the relatively short-lived peak of the storm surge. These better-performing sections were levees comprised of compacted, clayey soils; soils known to have far higher intrinsic resistance to erosion (see Chapters 9 and 10.)

We know a great deal about the soil types, and the placement and compaction conditions, that lead to differing types of performance with regard to erosion. Moreover, we are now increasingly able to perform specific tests of these materials, and to make reliable engineering assessments of expected behavior with regard to erosion (see Chapters 9 and 10.)

Important lessons here include the following:

2. *The use of materials excavated from the adjacent shipping channels resulted in some initial cost-savings, but these minor cost-savings were multiple orders of magnitude less than the subsequent damages that occurred when these levee sections failed. **Short-term cost savings in construction need to be balanced against the consequent increases in risk (the consequent reduction in likely reliability) for the resulting built system.***
3. *Highly erodeable embankment (and foundation) materials represent an intrinsic hazard, and their use should be avoided in flood protection systems defending significant populations.*
4. *When the use of such materials cannot be avoided, then great care should be taken to protect the sections by means of internal cut-offs, filters, and slope face protection (armoring) on the front and back faces and on the crest as well. Even then, the use of erosion-resistant soils is to be preferred if at all possible.*
5. *Levees (and composite levee/floodwall sections) can be designed to safely withstand some degree of overtopping, and for some period of time. Hurricane storm surges, unlike river floods, typically have their “peak” over only a limited number of hours. Given the economic challenges of designing flood protection systems for very high levels of*

(infrequently occurring) storm loading; the alternative of designing flood protection systems to perform safely without admitting any water into the protected areas for an fairly high levels of loading, but with sufficient resilience that they can be “overtopped safely” (overtopped for a while during a peak storm surge, but not erode and fail catastrophically) so that the system will continue to provide protection as the peak of an unusually large storm surge passes and then subsides might also be considered. Some water would enter the protected area(s), but the amount would be limited and it could be pumped out afterwards with minimal risk to life, and manageable property damages.

At some locations (e.g. adjacent to large concrete navigation gate structures) the use of the lightweight shell-sands was deliberately specified in order to minimize differential settlements and “gapping” at the contact between the concrete gate structures and the levee embankment. Prevention of this differential settlement by use of these highly erodeable materials led to massive eroded breaches at the south side of the bayou Bienvenue gate structure, and at the north side of the bayou Bienville structure along the MRGO frontage; an example of solving one problem while exacerbating another. Thus

6. *The consequences of any engineering decisions need to be considered on a system basis; there is a long history of engineering failures based on unintended consequences.*

Another disturbing issue was the fact that many sections of the regional flood protection system had levee crest elevations, and concrete floodwall elevations, that were below intended design grade. This was largely a result of the 40-plus years that the design and construction of the system had been underway, and the fact that benchmarks and datums for elevation control had progressively decreased in elevation as part of large-scale overall regional subsidence over that extended period of time. As a result, many sections of the regional system had crest heights and floodwall heights as much as 1 to 2 feet below intended design grade. [An excellent treatment of this issue with regard to datums and regional subsidence is presented in the IPET Draft Final Report; IPET, June 1, 2006.] This “loss” of levee crest and floodwall height exacerbated problems associated with overtopping.

In addition, the critical levees along the MRGO frontage at the northeast edge of the St. Bernard Parish/Lower Ninth Ward protected area were well below grade along much of their length. This was not an engineering lapse, nor a problem associated with subsiding datums. These levees were being constructed in stages, to allow for settlements and consolidation (to increase the strengths of the foundation soils prior to adding the next stage of levee embankment fill.) At the time of Katrina’s arrival, the USACE had long been requesting funds to place the final stage of fill along this frontage. Now it is too late.

It can be argued that this represents a tragic example of the intrinsic risk associated with over-long project durations for these types of massive, regional scale projects. Also that both White House and Congressional attention lapsed, and funds that could have been provided to complete these important levees were instead deferred, as issues elsewhere drew more urgent attention. Thus

7. *If we resolve to create and operate important flood protection systems to defend large populations, then we should commit sufficient funds and diligence to consummate this*

construction within a reasonable time span. Otherwise: (1) we are leaving populations at risk, as partially completed protection is no protection at all, (2) we invite problems that will naturally arise as a result of over-segmentation of discrete project elements that must perform perfectly well together as a contiguous system, and (3) overall system coordination and integrity will inevitably suffer as a result of progressive loss of institutional memory during the extended period of design and construction.

Finally, both of these levee frontages were “backed” by lower height secondary levees that were rapidly overtopped by the massive flows passing through the long breaches in the frontage levees. Indeed, the frontage levees failed so rapidly, and so early, that they did little to blunt the storm surge. The secondary levees were never intended to have to deal with an undiminished storm surge, and they were quickly overtopped along much of their lengths (though they were comprised of better, clayey materials and suffered admirably little damage at most locations from this massive overtopping.)

The lack of height of these internal “secondary” levees represented a wasted opportunity to provide defense in depth.

8. *As the regional system is now repaired and improved, further raising of these secondary levees would provide a potentially valuable second line of defense for the populous communities behind them. If the federal government will not fund this, then local interests should consider doing this on their own.*

These secondary levees are well-situated, and are currently comprised of good, erosion-resistant materials. The USACE has recently helped to raise the secondary levee across the middle of St. Bernard parish (the Forty Arpent levee) to Elev. +10 feet (MSL). These secondary levees should be considered part of a system along with the frontage levees, and with the ridges of high ground within their protected areas as well. The frontage levees should be designed to safely resist a considerable level of storm surge. In the unlikely event that an even greater storm surge exceeds this level, then they should be designed to overtop without eroding catastrophically, and the secondary levees should be designed and sized to entrap and water overtopping the frontage levees and so protect the populous communities. That would require coordination of levee heights, and likely of oversight agencies as well.

This, in turn, represents an example of an element of engineering that was sadly missing throughout much of the regional flood protection system; the asking of the vital question: “What if?”

There was a persistent lack of ductility and resilience throughout the regional flood protection system. Over and over, system elements and sections were designed to some specified level of loading, but no thought was given as to what would happen if that level was exceeded. Over and over, we studied sections that failed catastrophically, in a brittle manner, when for little or no increase in cost the section designs could have been modified to safely accommodate some minor exceedance of the design loading conditions, and where similarly relatively minor modifications could have rendered even more severe exceedances of the specified design loads at least far less devastating in terms of consequences for the protected communities. To a large degree, this failure to ask the important “what if?” question is a

function of the rules and regulations that govern the creation of these large systems. This needs to change.

9. *Instead of working to pedantically prescribed “design levels” mandated by Congress, the USACE should be allowed (and encouraged) to constantly ask the important “What if?” question. When that leads to the awareness that minor additional effort and expense would likely result in massive improvement in overall system performance and reliability, then a feed-back mechanism should be established to allow advantage to be taken of this.*

That would be an invaluable step forward, in many areas of federal operations.

11.3.4 The Central Region; the IHNC and the GIWW/MRGO Channel Frontages

The storm surge that swelled the waters of Lake Borgne was driven west along the east-west trending shared GIWW/MRGO channel to the “T” intersection with the IHNC, raising water levels within these channels and resulting in overtopping at many locations along both banks of both of these channels. Despite this overtopping, the performance of many of the levees and floodwalls along these channels was excellent. Several major failures occurred, but most of these were not caused by overtopping, but were instead the result of other issues as described below.

Along the east-west trending GIWW/MRGO channel, overtopping produced minor to moderate erosional distress at a number of locations, but no full failures (breaches) of full height earthen levee embankments occurred. The levees along both banks of this channel appeared to be comprised primarily of compacted, clayey soils, and the good performance of these materials in the face of moderate overtopping was encouraging.

Two breaches occurred along the north bank of this channel, at Sites #4a and #4b in Figure 11.4 (and Table 11.1.) The first of these (Site #4b) was a breach at an inadequate “transition” between a long reach of full height earthen levee as it joined (abutted) a mid-rise earthen levee reach with a sheetpile-supported concrete floodwall at its crest. The transition between these two adjacent project sections was a simple sheetpile wall, of lesser height than either the earthen levee to the west, or the floodwall section to the east. As a result, overtopping was most severe over the top of the short sheetpile wall transition section, and this overtopping preferentially eroded a deeply scoured trench at the rear side of the sheetpile wall. This, in turn, reduced the lateral support at the back side of the sheetpile wall section, and the laterally unbraced sheetpile wall was then pushed sideways by the storm surge water pressures on its front (outboard) side, and it failed.

This was one of many examples of inadequate detailing of “transitions” between adjacent major project elements that resulted in poor performance, and breaches at a number of locations. Thus

10. *“Transitions” where adjacent project elements join together were routinely problematic throughout much of the regional flood protection system. Successful design and construction of two adjoining project sections counts for little if the connection between*

them is not also successfully consummated. Significantly more attention needs to be paid to these “transitions”.

The second breach along this frontage (Site #4a) was a failure of a concrete floodwall. This floodwall was mainly a simple sheetpile-supported I-wall, but it had two short sections of T-wall with battered piles to provide increased rotational and lateral support. The T-wall sections performed well, but major lengths of the I-wall did not. The walls were overtopped, and the water cascaded over the walls and eroded trenches at the back sides of the walls. This, in turn, laterally unbraced the walls and some sections were laterally displaced while other sections rotated, some to a nearly fully horizontal position.

Like the situation described at Site #4b above, this overtopping did not have to result in erosion and unbracing of the floodwall. Installation of splash pads, or other erosion protection, at the rear side of the walls to prevent erosion by the water passing over the tops of the walls would have represented a relatively minor additional expense (estimated at less than 5% of the overall project section cost). The USACE felt that splash pads were disallowed; they were instructed by Congressional edict to design for a specified water level, and to install splash pads would be to provide for a higher level than that which had been authorized. In hindsight, everyone regrets this dilemma. To its credit, the USACE has already undertaken (even prior to full authorization) to install splash pads behind I-walls and/or to replace I-walls with T-walls (which have their own splash pads as a result of their inverted-T shape.) Thus

- (9. Repeated): Instead of working to pedantically prescribed “design levels” mandated by Congress, the USACE should be allowed (and encouraged) to constantly ask the important “What if?” question. When that leads to the awareness that minor additional effort and expense would likely result massive improvement in overall system performance and reliability, then a feed-back mechanism should be established to allow advantage to be taken of this.*
- 11. Concrete floodwalls can be designed to be safely overtopped to some considerable degree, and advantage can be taken of this in design and construction of an improved overall regional flood protection system.*

Further to the west, the rise in water levels within the IHNC produced overtopping at numerous locations, and it produced a number of failures as well. The overtopping was not directly related, however, to the most significant of these failures.

Two of the largest failures during hurricane Katrina were the pair of failures on the east bank of the IHNC, at the west end of the Lower Ninth Ward (Sites #6a and 6b.) These two major breaches were studied in detail, as presented in Chapter 6; Sections 6.3.1 and 6.3.2.

Although overtopping occurred along much of this IHNC frontage, and although this overtopping produced erosion at the inboard base of the concrete floodwall (and I-wall section) at the location of the south breach, both sections failed as a result of underseepage rather than overtopping.

The large south breach, hundreds of feet in length, failed as a result of underseepage-induced pore pressures which weakened the foundation soils beneath the inboard toe of the levee embankment, and resulted in translational stability failure of the embankment section (pushed laterally by the risen waters in the canal.) The northern breach, which was a narrower, deeper failure, was the result of underseepage-induced hydraulic uplift (“blowout”) at the inboard toe and underseepage-induced toe erosion and piping.

These conclusions contradict the findings of the IPET Draft Final Report of June 1, 2006, which found the failure of the south section to be the result of overtopping, scour at the inboard toe of the floodwall, and resultant lateral unbracing of the floodwall. The northern failure was attributed to deeper-seated semi-rotational failure of the foundation, primarily through a layer of soft clays, and this failure was assumed to have occurred surprisingly early, in order to explain observations of large amounts of water collecting on the inboard side along this frontage. The IPET report also mentions that underseepage-induced failure mechanisms were not studied, as the foundation soils were too impervious.

There was a long history of underseepage-related problems along this frontage, and it is likely that the IPET study would have pursued these if they had been so informed. Instead, they hued to the history of the local New Orleans District of the USACE, and “absolved” underseepage as a potential failure mechanism at these two important sites without bothering to perform formal analyses to study the possibility.

The IPET analyses of their preferred failure modes make no technical sense, and defy the available data regarding the strengths and stiffnesses of the soils involved (see Sections 6.3.1 and 6.3.2.) Moreover, there is a long history of problems associated with underseepage along this frontage, including both citizen issues with ponded waters and contractor’s difficulties with dewatering for construction. The most stunning demonstration of the high lateral permeability of the “marsh” deposits at this location, however, is a visually spectacular reverse crevasse splay (see Figure 6.45) produced beneath the temporary repair embankment section by the relatively small reverse flow gradients as the protected had nearly fully drained.

As shown in Figures 6.24 and 6.47, the relatively short sheetpiles at these two sections failed to achieve adequate cut-off of underseepage flow through “marsh” strata that were tantalizingly just below the bases of these sheetpile curtains. That was a repeated theme in this event, as underseepage-induced failures, as a result of inadequate sheetpile depths (where moderate extensions of the sheetpiles would likely have prevented failure) also occurred at the two major, and devastating, breaches on the London Avenue drainage canal (Sites #10 and #11a), and an additional underseepage-induced “incipient” failure began to develop on the east bank of the London Avenue canal (Site #11b) but halted as the section on the west bank failed first, drawing down the water levels.

Inadequate sheetpile depths, as a result of overly optimistic assumptions regarding the permeability of foundation soils, were thus found at five of the most important sites, including four of the most damaging breaches that occurred during hurricane Katrina. It is time for the New Orleans District to come to grips with the potential severity of the underseepage problem. This is a major issue, not only because it represents a likely

remaining source of potential vulnerability throughout the remainder of the system, but also because it may obviate current cost projections for further improvement of the regional system. The types of steps needed to remedy underseepage-related vulnerability are very different from the types of actions already being undertaken to reduce overtopping vulnerability.

As mentioned above, the USACE has already taken major steps to add concrete splash pads and/or to replace concrete I-walls with T-wall sections. This was laudable, and a very useful step with regard to addressing potential vulnerability associated with overtopping. It does not, however, also address the potential vulnerabilities associated with underseepage. At a recent press briefing a USACE representative at the IHNC east bank, at the west end of the Ninth Ward, indicated the massive concrete splash pads newly installed and remarked that “King Kong himself could not come over the top of that wall”. Unfortunately, Katrina did not so much come over the top of that wall, as she passed beneath it.

Important lessons here include:

- 12. It is important not to exonerate any failure mechanism(s) a priori, not before thorough analysis. This is true both in design, and in forensic investigations. In all cases, and especially in design, all failure mechanisms must be considered potentially guilty until either proven innocent, or until mitigated by appropriate design provisions.*
- 13. Underseepage is one of the common modes of levee failure, and it appears to represent a considerable potential source of vulnerability throughout much of the New Orleans regional flood protection system. In addition to five sites studied in detail because failures (or incipient failures) due to underseepage occurred there, numerous additional sites were reviewed in a cursory manner and our investigation team was routinely struck by the surprisingly shallow depths of the sheetpile curtains, and the manner in which potential concerns regarding underseepage appeared to have been wished away during design. The installation of massively longer (deeper) sheetpiles, often 60 feet in length and more, replacing original sheetpiles less than thirty feet in length, as part of repairs at numerous breach sites represents a de facto and unusually frank admission as to the systemic inadequacy of pre-Katrina sheetpile penetrations. This is a potentially serious source of continuing risk to the regional system, and it may also obviate current cost projections for upgrading the regional system (and recent appropriations for this purpose as well.)*
- 14. There is an urgent need to perform a thorough, system-wide review of potential underseepage-related vulnerability.*

In addition to the two major breaches at the east bank of the IHNC (at the west end of the Lower Ninth Ward), three additional breaches occurred on the west bank of the IHNC. These three breaches are Sites #5a, b and c. These were the first breaches to admit floodwaters into the main (downtown) protected basin. None of these three breaches managed to erode or scour a path back to the IHNC with a base consistently below sea level, however. Accordingly, although these breaches admitted floodwaters briefly while the water level within the IHNC was elevated, all three breaches subsequently ceased inflow as the storm subsequently subsided only a few hours later.

Site #5a was the west bank of the CSX railroad crossing. The failure at this site is particularly galling, as this site also failed during hurricane Betsy in 1965, so the repeat failure represents a very disconcerting failure to learn.

The rail crossing is part of a complex “penetration” through the federal IHNC frontage levees, as an adjacent roadway serving outboard side Port facilities crosses over the top of the federal frontage levee adjacent to the railroad line. Our investigation team was unable to learn just exactly who was overall in charge at this complex site with multiple overlapping jurisdictions; and that is a problem.

The rail line passes through a concrete T-wall structure with a rolling steel floodgate, so that the floodgate can be closed during storms to complete the perimeter frontage protection. Unfortunately, the steel gate had been damaged by a railroad accident several months prior, and it had been taken away for repairs. Accordingly, a temporary “sandbag levee” was erected across the missing floodgate opening; this washed away at some stage during hurricane Katrina. It is not clear who was in authority here, but the decision to remove the steel floodgate and allow trains to continue to operate, rather than affixing the damaged gate in place until it could be replaced, placed the entire community of the main (downtown) New Orleans protected basin (a population of approximately 250,000+) at risk. In hindsight, this was a difficult decision to justify.

Fortunately, the missing gate was not a principal source of flooding for the main (downtown) protected area. The main breach at this location was actually the result of composite action of the railroad embankment and the adjacent roadway section. Water appears to have passed first through the pervious gravel ballast at the top of the railroad embankment (representing the local “low spot” with regard to stopping flow), and it then eroded and undermined the adjacent roadway, resulting in a full breach. The levee embankment underlying the roadway appeared to consist in part of highly erodeable sands and shell-sands, and the presence of such highly erodeable soils without cut-off or other provisions to prevent catastrophic erosion was very ill-advised.

Lessons here include:

- 15. Someone needs to be overall in charge at “penetrations” and “transitions” where multiple groups and functions intersect, and where overlapping responsibilities result. Whoever is in charge needs both to be made responsible for the overall situation, and they need to be granted adequate authority as to successfully execute that responsibility.*
- 16. The continued operation of trains cannot be allowed to be considered more important than the safety of major urban populations.*
- (7, repeated.) Highly erodeable embankment (and foundation) materials represent an intrinsic hazard, and their use should be avoided in flood protection systems defending significant populations.*
- (8, repeated.) When the use of such materials cannot be avoided, then great care should be taken to protect the sections by means of internal cut-offs, filters, and slope face protection (armoring) on the front and back faces and on the crest as well. Even then, the use of erosion-resistant soils is to be preferred if at all possible.*

The other two breaches along this frontage occurred at the south end of the main Port of New Orleans. Both breaches occurred in full height earthen levee embankment sections that were comprised entirely of lightweight shell-sand fill. This was shocking to our investigators, and the lessons here are simple. Again

- (7, repeated.) *Highly erodeable embankment (and foundation) materials represent an intrinsic hazard, and their use should be avoided in flood protection systems defending significant populations.*
- (8, repeated.) *When the use of such materials cannot be avoided, then great care should be taken to protect the sections by means of internal cut-offs, filters, and slope face protection (armoring) on the front and back faces and on the crest as well. Even then, the use of erosion-resistant soils is to be preferred if at all possible.*

An additional set of partially developed erosional features occurred at a number of locations on the east bank of the IHNC at the west edge of the New Orleans East protected area. These are grouped together as “Site #7” in Figure 11.4 (and Table 11.1.) Overtopping occurred along much of this frontage, but the erosional distress systematically occurred at “transitions” between adjacent, disparate system elements (e.g. at transitions between full height earthen embankments and adjacent gated concrete floodwall segments, etc.) Here, again, it was transitions rather than the main segments themselves that proved problematic.

Our field investigation team were initially puzzled that these multiple features all appeared to be partially developed erosional features, on their way to failure but failing to reach full failure. As our studies progressed, however, it became clear that the lands on the inboard side were already filling with floodwaters as these features were developing, and this reduced the gradients and the durations of flow. It is not possible to know whether any of these features might have developed into full breaches if the inboard side lands had been more successfully defended against flooding from breaches that occurred at other locations. Lessons here thus include

- (10, repeated.) *“Transitions” where adjacent project elements join together were routinely problematic throughout much of the regional flood protection system. Successful design and construction of two adjoining project sections counts for little if the connection between them is not also successfully consummated. Significantly more attention needs to be paid to these “transitions”.*
17. *The multiple transitions along this frontage suffered erosional damage but did not fully fail (and breach. But they may not have been fully tested as floodwaters were likely already rising on the inboard side lands as a result of massive breaches at other locations. These transitions should therefore be thoroughly re-evaluated as part of ongoing flood protection system upgrades.*

11.3.5 The Lake Pontchartrain Frontage, and the Drainage Canals

As the eye of the hurricane finally passed to the northeast of New Orleans, its counter-clockwise swirling winds drove a final storm surge south along the shoreline of Lake Pontchartrain, along the north edge of the city.

This final storm surge produced some degree of overtopping at several locations along the lake frontage levees of the New Orleans East protected area, and two failures occurred (Sites #8a and b.) Site #8a was another complex “penetration” where multiple rights of way passed, together, through the federal levee perimeter. These included an elevated State highway, yet another railroad line, and a ground-level roadway. These three elements interacted poorly together; flow through the pervious railroad embankment ballast undermined the connection between a concrete floodwall protecting a support for the elevated highway and the adjoining surface roadway, and flow across these features also eroded an adjacent section of the Federal perimeter earthen levee. Once again, it was not clear who, if anyone, was overall in charge at this complex site. Thus

(15, repeated.) Someone needs to be overall in charge at “penetrations” and “transitions” where multiple groups and functions intersect, and where overlapping responsibilities result. Whoever is in charge needs both to be made responsible for the overall situation, and they need to be granted adequate authority as to successfully execute that responsibility.

The second site was a long section of floodwall whose crest was surprisingly low. Overtopping occurred along this low section over approximately a mile of floodwall length, despite a lack of persistent, sustained overtopping at any adjacent sections along this lakefront frontage. Significant scour occurred at the rear base of this floodwall, and a minor breach occurred at one location where floodwall panels shifted a bit as a result.

Farther to the west, the storm surge along the Pontchartrain Lake frontage levees at the north end of the main (downtown) New Orleans protected area did not produce meaningful overtopping along the lake frontage. This storm surge did, however, raise the water levels in three drainage canals that emptied into the Lake.... and three major breaches occurred along these drainage canals. These three breaches all rapidly scoured to well below sea level, and as a result floodwaters continued to flow in through these breaches for three days (even after the storm surge had subsided) eventually equilibrating with the still slightly elevated waters of Lake Borgne on Thursday, September 1st. These floodwaters infilled much of the main (downtown) New Orleans protected basin, resulting in roughly half of the overall deaths during hurricane Katrina, and a similar fraction of the damages as well.

The three drainage canals should never have been exposed to storm surge rise. The USACE had fought for years to install storm gates at the north ends of the three canals, but had been defeated (outmaneuvered in Congress) by local interests as a result of dysfunctional interactions and distrust between the local Levee Board (who were nominally responsible for perimeter levee protection) and the local Water and Sewerage Board (who were responsible for pumping and “unwatering” of New Orleans.) Every drop of rainwater that falls into New Orleans has to be pumped out, as the city is largely below sea level. Rainfall, and constant levee underseepage, are the principal concerns for “unwatering” on the part of the Water and Sewerage Board in most years, while the Levee Board is concerned primarily with providing protection during infrequent river floods and hurricanes. Accordingly these two organizations have differing principal focuses.

That led to dysfunctional interaction between them, distrust, and eventually even animosity. The Water and Sewerage Board, concerned that storm gates would be “perimeter protection” under the control of the Levee Board (and thus might possibly not be opened promptly when rainfall required pumping out through the drainage canals) fought successfully to have Congress decline the USACE’s request for construction of storm gates at the heads of the canals.

Unfortunately, the construction of the floodgates would have been the superior technical solution. Instead, the canals remained “open” to Lake storm surges, and the three canals thus represented daggers pointed at the heart of New Orleans.

The USACE then attempted to exempt the three drainage canals (the 17th Street canal, the Orleans canal, and the London Avenue canal) from federal responsibility. Local interests again outmaneuvered the Corps, and Congress specifically declared these to be a federal responsibility; they required the USACE to raise the levels (elevations) of protection along the sides of these three canals.

The USACE correctly pointed out that the “footprints” available for levees along the sides of these canals (especially the 17th Street and London Avenue canals) were insufficient, as homeowners’ properties abutting the canals encroached on the existing levees; in some cases property lines extended up the levee slopes to the edges of the narrow crests. There was insufficient room available to safely widen these levees in order to add to their heights.

This too was over-ruled, and the USACE was directed to raise the levels (elevations) of protection along these canals, within the existing (inadequate) “footprints” available. The results were catastrophic. Lessons here include

18. *The USACE is the lead Federal agency with expertise regarding levees and flood control. The USACE needs to be resolutely vocal and persistent in declining to undertake actions that it considers to be unsafe. Congress needs to better heed due warning from the Corps. The interactions between Congress and the Corps need to involve improved give-and-take; the Corps needs to be allowed to better assert strongly held professional opinions.*
19. *Local interests, and special interests, cannot be permitted to “outmaneuver” legitimate technical concerns with regard to Public Safety. The local Levee Board, and the local Water and Sewerage Board, should have been required to resolve their personal differences in the greater interest of Public safety.*
20. *The USACE felt that the path they were directed to follow was unsafe. They could have simply refused to take that path. That might have required resignations on the part of the leadership; those would have been honorable resignations.*

Having been essentially ordered to raise the levees (and floodwalls) along the three drainage canals, the USACE recognized that this posed significant technical challenges. Accordingly, they next performed a very well-directed full-scale experiment in the nearby Atchafalia River basin (on soil conditions very closely mirroring the challenging geology of

the drainage canals) in which a concrete floodwall (I-wall) was modeled using a plain sheetpile wall. This test section (the E-99 test section) was constructed on the berm of an Atchafalía levee, with the berm height closely modeling the existing levee heights along the three drainage canals. A sheetpile cofferdam was constructed, and filled with water (to model storm surge loads against the sheetpile/floodwall).

This important large-scale field experiment clearly showed that under storm surge rise, a “gap” was likely to form between the sheetpile curtain supporting concrete I-walls, and that this gap would fill with water; significantly increasing the lateral pressures applied by water pressures against the sheetpiles/floodwalls. This mechanism subsequently figured in all three of the catastrophic drainage canal failures that occurred during hurricane Katrina, and in a number of other failures at other sites during Katrina as well. The failure to include this potential failure mode in the subsequent analysis and design of large elements of the regional flood protection system proved disastrous.

Unfortunately, the important lessons from this expensive and well-directed full-scale field test were never subsequently incorporated into the design of the floodwalls used to raise the protection elevations along the three drainage canals. Thus

21. Our investigation uncovered a persistent failure to learn; to adapt to technical advances, and even to heed the results of the USACE’s own research, on the part of the New Orleans District. Outdated analysis methods, and strongly held views (which proved to be in error) were key failings in the design and construction of the flood protection system at a number of locations.

Another particularly important location was the south end of the Orleans drainage canal (Site #9.) Although it was located between the 17th Street drainage canal and the London Avenue drainage canal (catastrophic breaches occurred on both of these canals), no breaches occurred on the Orleans canal. Instead, storm surge waters simply flowed freely into the heart of New Orleans through an unfinished “gap” in the floodwalls lining this canal. A section of concrete floodwall approximately 200 feet in length at the south end of this canal was “omitted”; rendering the miles of floodwalls lining the remainder of the canal somewhat superfluous.

The omission of the last several hundred feet was done to protect the ancient (1904) brick building housing the several giant Woods pumps that pumped waters from the neighborhood into the canal (and thus into Lake Pontchartrain.) This brick building forms a “T” at the south end of the canal, closing the canal. When the canal water levels rise more than about five or six feet (e.g.: during pumping) , water seeps actively through the walls of the old brick building, and it is clear that significantly further rises in water levels would threaten to buckle the wall. Thus, either: (1) the Levee Board (who were responsible for “protection”) would have had to erect a barrier to protect the Water and Sewerage Board’s pump house , or (2) the Water and Sewerage Board would have had to expend their own resources to erect this protection themselves; helping out the Levee Board in the process by closing the end of the canal. As a result of internecine battling between these two agencies, and their inability to resolve their differences in the interest of the greater common good (and Public safety), neither occurred. Instead a gap was left in the floodwall (to control maximum

possible canal water elevations (at Elev. +9 feet) and a “spillway” section was constructed across this gap until the matter could be further resolved.

(19, repeated.) Local interests, and special interests, cannot be permitted to “outmaneuver” legitimate technical concerns with regard to Public Safety. The local Levee Board, and the local Water and Sewerage Board, should have been required to resolve their personal differences in the greater interest of Public safety.

Eventually, however, floodwall systems were designed and constructed to raise the crest elevations of the levees along these three canals. A number of engineering errors, poor judgements, and poor decisions occurred during this process, and these too contributed to the three catastrophic breaches that occurred (at Sites #10, 11a, and 12b.) In addition, two “incipient” failures nearly occurred, but which were “saved” by nearby failures that rapidly drew down the canal water levels. One of these “near failures”, on the west side of the 17th Street canal (Site #12a) would have resulted in flooding of a considerable portion of heavily populated Jefferson parish, and would have significantly increased the overall damages (and likely loss of life as well) from hurricane Katrina.

The first mistake was the failure to secure adequate right-of-way to widen the levees, to provide adequate embankment mass and weight as to sustain the increased lateral water forces that would be imposed by taller floodwalls atop the levee crests. This also meant lack of access and control over some of the inboard side levee faces and the critical inboard toe regions; rendering both inspections and necessary maintenance difficult.

These would both have disastrous consequences. Failure to purchase adequate right-of way contributed significantly to inadequate lateral stability at the massive breaches at the 17th Street canal (Site #12b) and at the west bank near the north end of the London Avenue canal (Site #11a.) Lack of control, and lack of access for inspection of the critical inboard toe areas led to rampant growth of trees at the toes (a known hazard), and even to excavations for swimming pools near the inboard toes of the levees in this critical region. This uncontrolled growth of trees appears to have contributed to the large failure on the east bank of the London Avenue canal (Site #10.)

*(2, repeated.) The failure to purchase adequate right-of-way in what had become (expensive) developed neighborhoods resulted in initial project savings, but these cost-savings were multiple orders of magnitude less than the subsequent damages that occurred when these levee sections failed. **Short-term cost savings in construction need to be balanced against the consequent increases in risk (the consequent reduction in likely reliability) for the resulting built system.***

22. *The inboard toe region is a critical area with regard to both inspections and maintenance. Uncontrolled vegetation growth and other obstructions to maintenance and inspections need to be precluded. Large trees can die and leave rotted root systems that provide dangerous paths for seepage, and during hurricanes strong winds (and ground wetting which reduces root anchorage) routinely lead to toppling of trees. Trees on the inboard levee faces and the inboard toe regions can thus fall, leaving sudden voids that can cause or exacerbate “blowouts” and/or erosion and piping failures. Conditions*

on the inboard side slopes and toes of considerable lengths of the levees lining both the London Avenue and 17th Street canals represented clear potential hazards in these regards.

23. *In addition, it is customary policy for the USACE to require serviceable crest roads at the tops of levees to provide access for inspection, maintenance and emergency repairs. Given the failure to acquire adequate right-of-way, and resulting narrow crest widths, this was waived.*

The failure at the 17th Street canal was a lateral translational failure of the levee embankment, with the principal shear surface constrained by a thin, weak, and highly sensitive layer of organic clayey silt. Only one to several inches in thickness, this layer resulted from a previous hurricane that passed through; churning up organic matter, mixing it with the local silts and clays, and depositing a layer heavily flocculated clayey silt due to the storm-induced temporary increase in salinity of the local waters. This layer, which was only one to several inches in thickness, was well-hidden by an overlying layer of sticks and twigs and leaves, representing storm-blown detritus from the causative hurricane. This overlying layer obstructed conventional geotechnical sampling of this thin, sensitive layer, and also clear detection of this layer by conventional CPT.

The presence of this critical stratum went undetected by the original design studies, and by the post-event IPET forensic studies as well. The failure to detect this layer in both studies, despite drilling numerous boreholes through it, and pushing multiple CPT through it as well, was largely a result of employing “common practice” in which the field drilling (and CPT) were performed by personnel without special experience or geological expertise. The IPET team certainly had expert geological engineers, with experience with these types of strata, who could have usefully advised this process, but they were sidelined with other tasks (including writing up “geology” sections for the report.) There was segmentation (or compartmentalization) of the work both in the original design studies and in the subsequent forensic studies. Field personnel performing the drilling were insufficiently directed by engineers who had performed the important initial post-event forensic inspections, expert geologic input was insufficient, and the eventual analysts were not properly appraised of the full pertinent details by the other sub-teams.

That contrasts sharply with the approach taken by our (ILIT) team at this site. Despite considerable prior experience with these types of deposits, a thorough study was made of local geological nuances prior to beginning drilling and sampling (and CPT). Expert senior team members were present at the field boring and sampling, and the CPT, including specifically top-level expertise in geological engineering. Careful initial (immediate post-event) field forensics had already led to the suspicion that the failure was a lateral translational failure, controlled by a weak (and likely highly sensitive) layer occurring at a depth of approximately 3 to 8 feet beneath the inboard toe, producing laterally exiting toe features (including exiting overthrusts) to unusually great distances beyond the levee toe. Having studied the local geology, a highly sensitive organic clayey silt or silty clay layer (which might be very thin, and likely screened by overlying wind blow organic detritus) was a leading potential suspect. Our first boring discovered the failure stratum, and we then proceeded to follow it across the site (including sampling it at locations within the failure zone, at the toe of the displaced intact

levee block, where the layer was clearly uni-directionally sheared and remoulded; incontrovertibly the failure surface.)

The lessons here include:

24. *Engineering geology is of vital importance. Always has been, and always will be. It must be interwoven throughout all phases of geotechnical works; from pre-study, through site investigation, and through analysis and design as well. Geologists are too often treated as second class citizens, and some geotechnical firms no longer even “need” them. Failure to avail ourselves of expert geological insight, and at all stages of a project, is to needlessly imperil the effort.*
25. *Increasingly, the trend in “modern” practice is to segment geotechnical works; separating field investigation (e.g. borings and sampling, CPT, etc.), laboratory testing, analysis, and design. These elements need to be seamlessly interwoven, and iterative cross-communication between the personnel performing these needs to be thorough. Sadly, that is increasingly not the case; not only in government works, but in common (private) civil practice as well. This segmentation can be more “cost effective”.... The risk is that something will be missed.*

In addition, review of the original design studies showed ten additional engineering lapses and/or questionable judgements at this site, as enumerated in Chapter 8; Section 8.3.7.1(c). These included extrapolation of data across excessive distances, failure to recognize “red flags” such as failure to recover samples at the same elevation in nearby borings (the elevation where the critical, sensitive, and very-difficult-to-sample organic clayey silt stratum occurred), etc. Readers are directed to this section for a full listing. Several key lessons include the following:

- Basic principles of soil mechanics were neglected, as the influence of increased effective stress beneath the centerline of the levee embankment was ignored, and soil shear strengths beneath the levee toes (where effective overburden stresses were smaller) were overestimated as a result. Shear strengths were extrapolated over lateral distances that were too great, and sometimes over excessive vertical distances. Shear strength profiles used for design calculations were not well-justified at certain, critical, elevation ranges by the data available.
- An archaic analysis method, the Method of Planes, was used for most stability calculations. This method (involving three blocks or wedges, and a conservative side force assumption between wedges) provides a demonstrated conservative answer for cases to which it can be applied. It is inflexible with regard to geometry, however, and was unable to deal with non-level stratigraphy and curvilinear failure surfaces. More modern and flexible methods were in common use at the time of these design studies, and should have been employed.
- The formation of a water-filled “gap” between the sheetpile curtain and the outboard portion of the levee embankment was not considered among the potential failure modes; despite the well-directed E-99 test section full-scale experiment that had shown this mode to be of concern.

- The design Factor of Safety for overall lateral stability during “transient” storm surges was only 1.30. That was far too low to allow an adequate margin for errors and uncertainties. That design Factor of Safety had evolved from tradition, and dated back to the middle of the last century, at which time it was selected for design of levees providing protection for agricultural lands (not populous regions). The design standard had not been updated, nor adapted for levees protecting large populations.
26. *All of these problems would have been expected to be caught and challenged by a competent panel of independent technical reviewers. Instead, reviews of the largely locally “outsourced” engineering design were performed internally within the USACE. **The mobilization of suitable, independent expert review capability is one of the most important steps that can be taken to enhance the likelihood of improved system reliability and performance in future events.***
27. *There was a persistent failure to learn, and to adapt to technical advances, within the local New Orleans District that affected performance of the regional flood protection system at numerous sites. Difficulties in recognizing potentially important “new” issues has continued in some cases since the hurricane; “That’s not how we do it” needs to cease to be an issue.... On the heels of system-wide failure, changes are in order. The USACE needs to ensure that the New Orleans District is adequately technically staffed for the magnitude and technical difficulty of the challenges it faces with regard to the engineering design and construction of critical flood protections systems in a region with exceptionally challenging geology, and in the face of both local and federal governmental assistance/interference as well. Suitable technical advances need to be studied, and embraced if appropriate. Upgrading personnel, and education and training, will also be important.*
28. ***Design standards, especially with regard to targeted levels of system reliability, need to be reconsidered (see Figures 11.1 and 11.2).** This is already underway; the USACE is performing a comprehensive re-assessment of design procedures and standards, and treatment of flood protection systems on a risk-based systems basis is anticipated to be an important element of this. That is a very promising development.*

Two additional large failures (and breaches) occurred on the London Avenue drainage canal. The failure on the east bank, near the south end (Site #10) was the result of underseepage-induced erosion and piping and/or underseepage-induced hydraulic uplift at the inboard toe (“blowout”), and it may have been exacerbated by a large tree at the inboard toe of the levee that blew over during the storm (at approximately the location of the failure.) The failure on the west bank, near the north end, was an underseepage-induced lateral embankment stability failure; the embankment slid laterally, pushed by the increased canal water pressures, and shearing occurred along foundation soils whose strengths had been reduced by underseepage-induced pore pressure increases (and resultant reductions in effective stress.)

The principal lessons to be learned from these two additional cases are repeats of lessons cited previously, and will not be repeated here again.

11.4 References

Christian, John T. (2004), “Geotechnical Engineering Reliability: How well do we know what we are doing?” *J. of Geotechnical and Geoenvironmental Engineering*, Vol. 130, No. 10, October 1, 2004.

IPET, (2006), “Performance Evaluation of the New Orleans and Southwest Louisiana Hurricane Protection System, Draft Final Report of the Interagency Performance Evaluation Task Force, Volume VI – The Performance – Interior Drainage and Pumping,” available online: <https://ipet.wes.army.mil/>, date accessed: June 1, 2006.

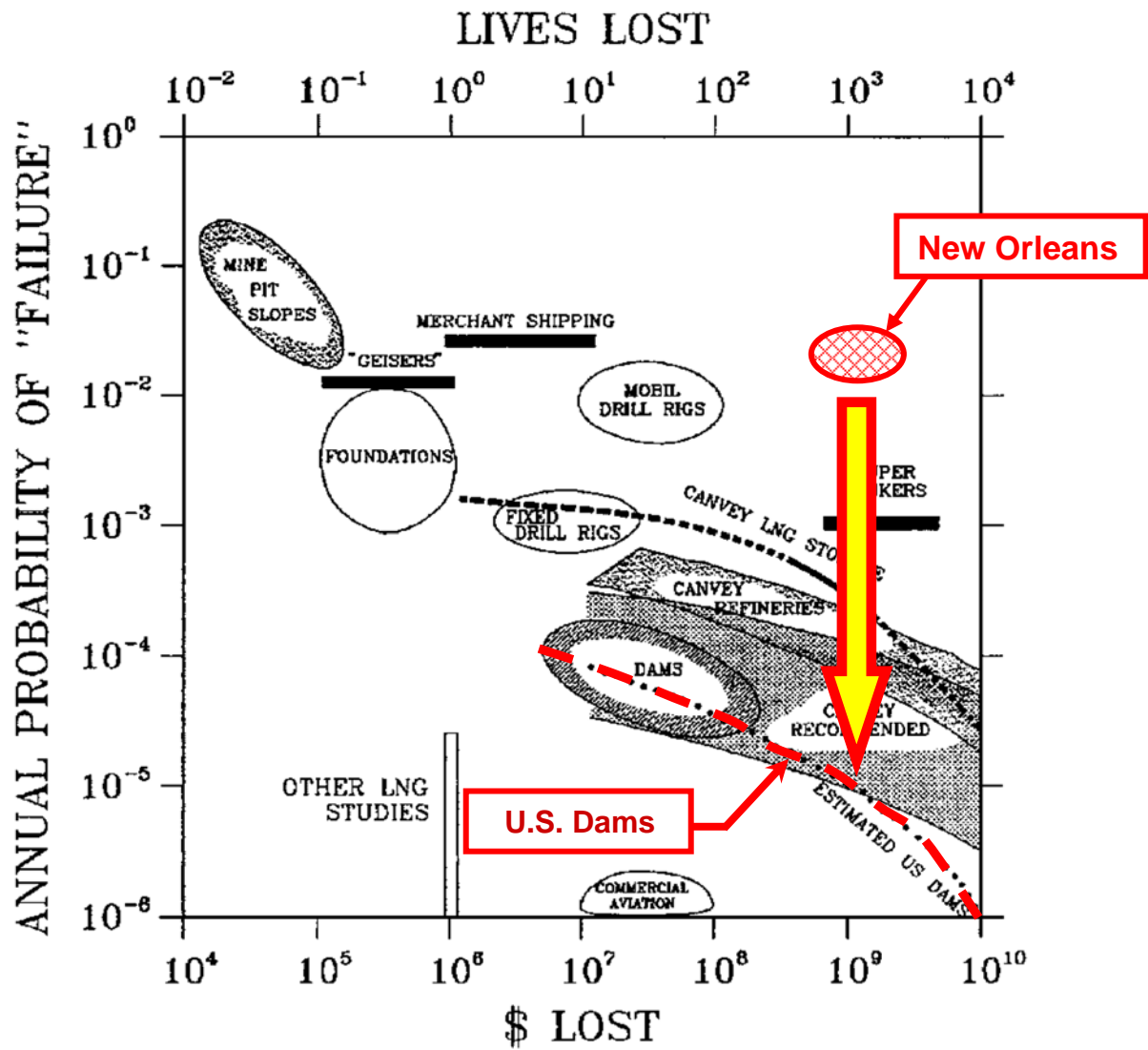


Figure 11.1: Risk plot showing the estimated pre-Katrina risk associated with the New Orleans regional flood protection system, and customary risk levels for current U.S. Practice with dams.

[Baseline Figure from Christian, 2004]

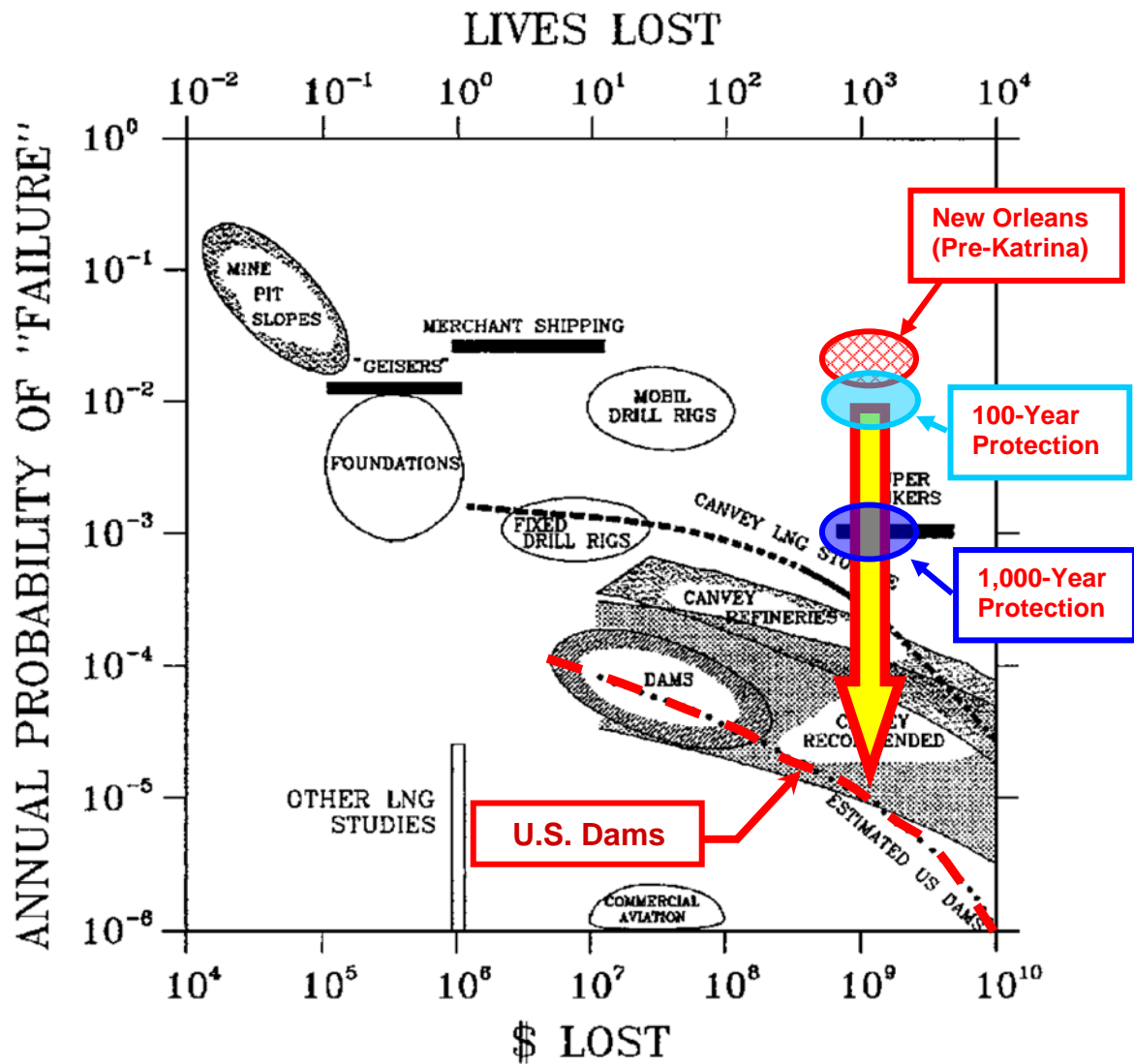


Figure 11.2: Risk plot showing the estimated pre-Katrina risk associated with the New Orleans regional flood protection system, and customary risk levels for current U.S. Practice with dams. Also shown are projected New Orleans risk levels for “successful” 100-year and 1,000-year storm and flood design.

[Baseline Figure from Christian, 2004]

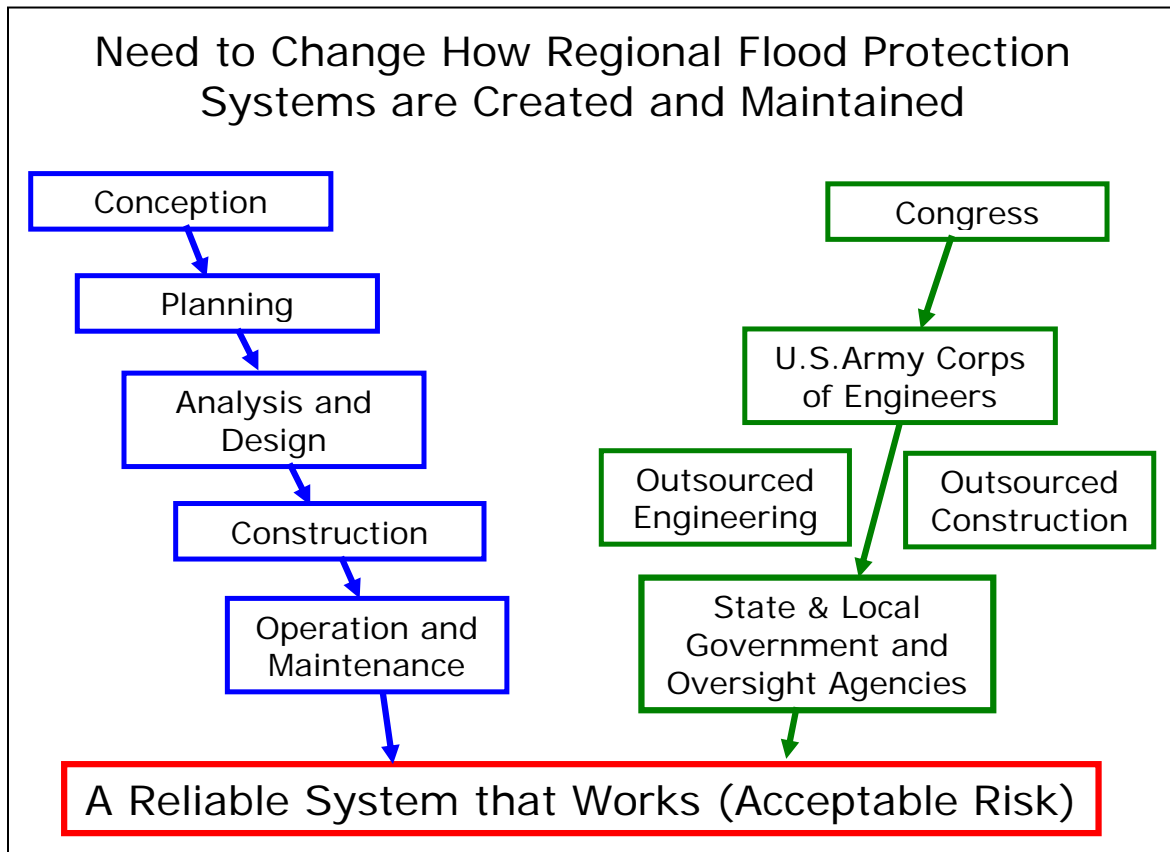


Figure 11.3: Engineering and organizational elements intrinsic to the creation, operation and maintenance of major U.S. regional flood protection systems in the Mississippi river basin.

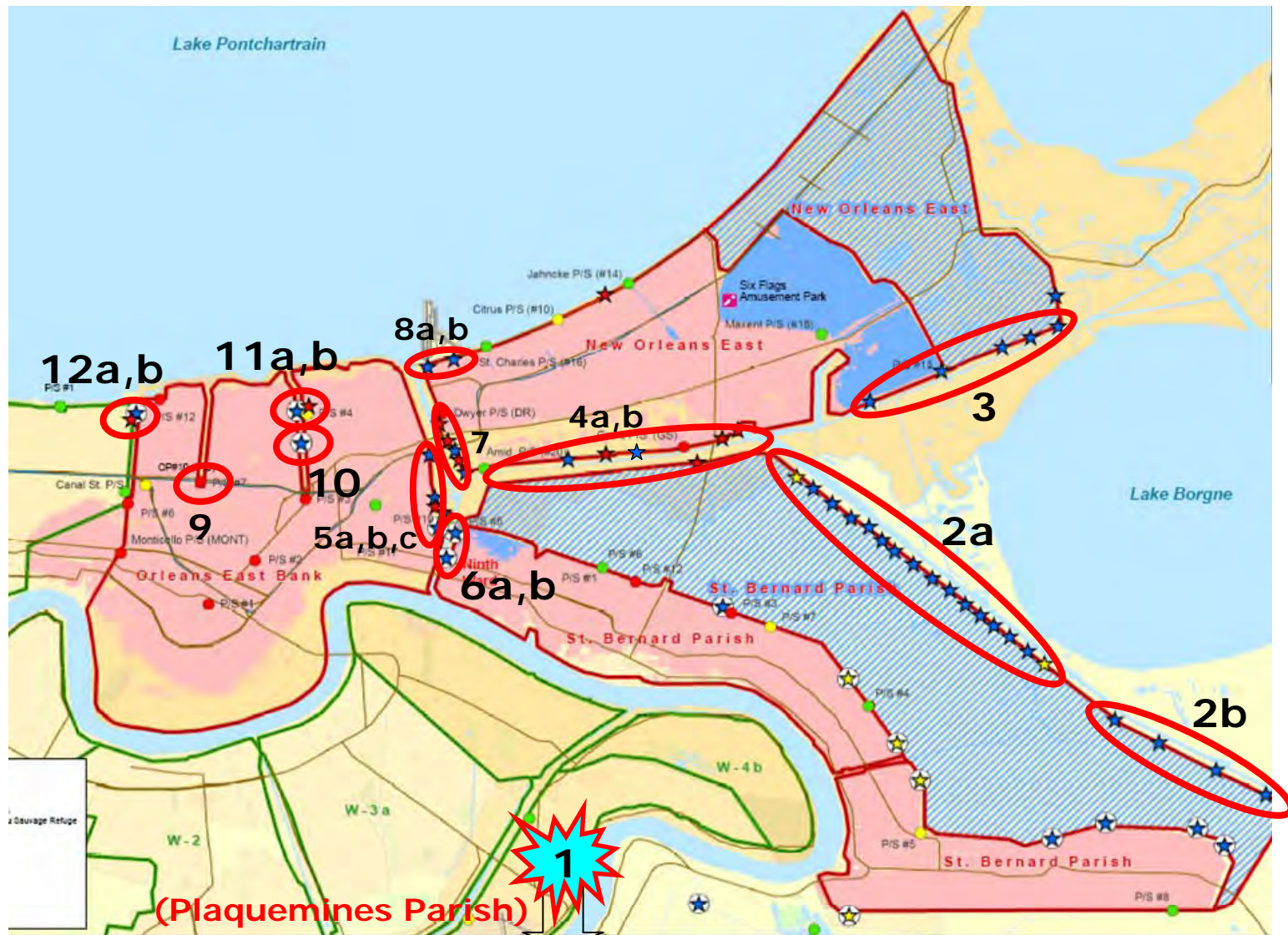


Figure 11.4: Summary of principal failures, breaches, and other locations of interest. (Blue stars mark breaches, red stars mark locations of distress.) [Base map provided by the USACE]

Table 11.1: Summary of Principal Damage Features Studied

Site No. or Group No.	General Location	Failure Mechanism and Cause (This Study: ILIT)	Failure Mechanism and Cause (IPET; June 1, 2006)	Severity of Consequences	Comments
1	Plaquemines Parish	Multiple failures resulting principally from massive and sustained overtopping.	Same as ILIT	Catastrophic	Storm surge and waves generally exceeded design levels.
2a	MRGO Frontage, St. Bernard Parish	Massive erosion and scour along many miles of levees due to waves, overtopping, through-flow, and use of highly erodeable embankment materials.	Overtopping erosion	Catastrophic	
2b	South MRGO Frontage, St. Bernard Parish	Erosion and scour along isolated sections of levees due to waves, overtopping, through-flow, and use of highly erodeable embankment materials.	Overtopping erosion	Catastrophic	
3	GIWW Frontage, southeast corner of New Orleans East	Massive erosion and scour along many miles of levees due to waves, overtopping, through-flow, and use of highly erodeable embankment materials.	Overtopping erosion	Catastrophic	
4a	North Bank of GIWW; the Citrus back levee floodwall	Overtopping of concrete I-wall, resulting in scour behind the wall, which was pushed sideways by elevated water levels.	Same as ILIT	Moderate	Consequences would have been more severe if other sections of the protected basin had not failed.
4b	North Bank of GIWW; sheetpile "transition" wall	Overtopping of concrete I-wall and sheetpile transitions resulting in scour behind the wall, which was pushed sideways by elevated water levels.	Same as ILIT	Moderate	Consequences would have been more severe if other sections of the protected basin had not failed.
5a	CSX Rail Crossing, west bank of IHNC	Poor coordination and poor interaction of multiple elements at a complex "penetration." Also, use of highly erodeable fill materials.	Rail gate absent	Moderate	
5b	Earthen levee embankment near south end of Port	Use of highly erodeable fill materials.	Overtopping erosion	Moderate	
5c	Second earthen levee embankment near the south end of the Port	Use of highly erodeable fill materials.	Overtopping erosion	Moderate	

Table 11.1 (cont'd)

Site No. or Group No.	General Location	Failure Mechanism and Cause (This Study: ILIT)	Failure Mechanism and Cause (IPET; June 1, 2006)	Severity of Consequences	Comments
6a	East bank of IHNC at edge of the lower Ninth Ward: South Breach	Underseepage-induced lateral transitional failure.	Overtopping of I-wall and scour	Catastrophic	
6b	East bank of IHNC at edge of the lower Ninth Ward: North Breach	Underseepage-induced erosion and piping and/or underseepage-induced hydraulic uplift at inboard toe.	Deep, semi-rotational foundation failure through soft clays	Catastrophic	
7	Cluster of minor erosion features at "transitions," east bank of IHNC	Erosion at inadequately detailed "transitions" between adjoining, disparate flood protection system segments.	- - -	Minor	Consequences might have been more serious if other sections of the protected basin had not failed.
8a	Erosional breach at northwest corner of New Orleans East	Erosional failure of another complex "penetration" where multiple interacted poorly as they crossed the perimeter levee system.	- - -	Moderate	Consequences might have been more serious if other sections of the protected basin had not failed.
8b	Overtopping and breach at floodwall behind Old Lakefront Airport	Overtopping of a surprisingly low floodwall, resulting in erosion behind the floodwall.	Same as ILIT	Moderate	
9	"Missing" floodwall section at south end of the Orleans Canal	Floodwall section omitted due to poor interactions between local oversight agencies.	- - -	Minor	
10	South breach on the east bank of the London Avenue drainage canal	Underseepage-induced erosion and piping and/or hydraulic "blowout" at the inboard toe.	Same as ILIT	Catastrophic	

Table 11.1 (cont'd)

Site No. or Group No.	General Location	Failure Mechanism and Cause (This Study: ILIT)	Failure Mechanism and Cause (IPET; June 1, 2006)	Severity of Consequences	Comments
11a	North breach on the west bank of the London Avenue drainage canal	Underseepage-induced lateral stability failure.	Same as ILIT	Catastrophic	
11b	Incipient failure on the east bank	Underseepage-induced lateral stability failure was beginning, when the west bank failure occurred and drew down the canal water level.	- - -	Negligible	This was nearly a fourth catastrophic failure along the drainage canals.
12a	Breach on the east bank of the 17 th Street Canal	Lateral translational levee foundation failure on a highly sensitive layer of organic clayey silt embedded within "marsh" deposits.	Deeper, semi-rotational foundation failure within soft clays	Catastrophic	
12b	Incipient failure on the west bank	Deeper, semi-rotational foundation failure within soft clays.	- - -	Near miss	This would have flooded large portions of the adjoining and heavily populated Jefferson parish.

CHAPTER TWELVE: ORGANIZED FOR FAILURE

We reflect on the 9/11 Commission's finding that the most important failure was one of imagination. The Select Committee believes Katrina was primarily a failure of initiative. But there is, of course, a nexus between the two. Both imagination and initiative - in other words, leadership - require good information. And a coordinated process for sharing it. And a willingness to use information - however imperfect or incomplete - to fuel action.

Hundreds of miles of levees were constructed to defend metropolitan New Orleans against storm events. These levees were not designed to protect New Orleans from a category 4 or 5 monster hurricane, and all of the key players knew this. The original specifications of the levees offered protection that was limited to withstanding the forces of a moderate hurricane. Once constructed, the levees were turned over to local control, leaving the USACE to make detailed plans to drain New Orleans should it be flooded.

The Local sponsors - a patchwork quilt of levee and water and sewer boards - were responsible only for their own piece of levee. It seems no federal, state, or local entity watched over the integrity of the whole system, which might have mitigated to some degree the effects of the hurricane. When Hurricane Katrina came, some of the levees breached - as many had predicted they would - and most of New Orleans flooded to create untold misery.

A Failure of Initiative

**Final Report of the Select Bipartisan Committee
U.S. House of Representatives, 109th Congress (2006)**

12.1 Introduction

This chapter summarizes results of studies performed by members of the Independent Levee Investigation Team (ILIT) into the organizational and institutional factors associated with failure of the Flood Defense System for the greater New Orleans area (NOFDS).

Over a period of eight months following failure of the NOFDS on 29 August 2005, the ILIT examined more than 2,800 documents, conducted more than 220 interviews, and reviewed more than 370 inputs from the general public. During the past 8 months, there have been a large number of extensive investigations into the reasons for the failure of the NOFDS. The ILIT made full use of results from these investigations. These results were combined with results from the ILIT investigations to formulate the primary findings documented in this chapter; organized for failure.

Chapter 13 outlines our thoughts on future organizational developments; *organizing for success*. Chapter 14 summarizes background on engineering a long-term NOFDS and the associated engineering guideline developments; *engineering for success*.

Appendix F presents a synopsis of the history of developments in the NOFDS between 1965 and 2005, summarizes background on understanding failures of engineered systems, and provides key quotations and results from other studies of failure of the NOFDS. Results from studies of the engineering and organizational aspects associated with future developments of a NOFDS are summarized in Appendix G. Appendix H, by Dr. Edward Wenk, Jr., documents a study of *How Safe is Safe? - Coping with Mother Nature, Human Nature and Technology's Unintended Consequences*.

12.2 Purposes

The ILIT studies have two purposes:

- to understand how and why the failure of the NOFDS developed, and
- to understand alternatives to reduce the likelihoods and consequences of such future catastrophes.

If we can adequately understand the mistakes of the past, then perhaps we have a chance to avoid making them in the future.

The ILIT approach in this study was to include historical and organizational - institutional issues, political and budgetary considerations, decision making, utilization of technology, and the evolving societal, governmental, and organizational priorities over the life of the NOFDS. One cannot develop an adequate understanding of the failure of the NOFDS without understanding both the engineering and organizational factors that were interwoven in development of this failure.

12.3 Failure of the New Orleans Flood Defense System

Of particular importance in this diagnosis is the organizational - institutional *Technology Delivery System (TDS)* that was used to develop the NOFDS. This TDS is comprised of three major components: (1) Government (federal, state, local), (2) Industry, and (3) the Public. All of these components and elements are interconnected through a complex series of multiple

connections that represent information and communication transmission. Inputs to the system include technical information, human and natural resources, capital, manufactured goods and services and values and preferences. Outputs from these components are represented in the NOFDS including its intended and unintended consequences.

The Government component is represented by agencies from all three branches (executive, legislative, judicial) at federal, state, and local levels. There are important multiple connections among federal, state, and local (parish, city) agencies. In the case of the NOFDS, the primary agencies are the Corps of Engineers, the Louisiana Department of Transportation and Development, and the parish levee boards and sewerage and water boards. All of these government agencies are interconnected with a multitude of other federal, state, and local agencies. These parts of the TDS were summarized by the Select Bipartisan Committee to Investigate the Preparation for and Response to Hurricane Katrina (2006):

Several organizations are responsible for building, operating, and maintaining the levees surrounding metropolitan New Orleans. USACE generally contracts to design and build the levees. After construction USACE turns the levees over to a local sponsor. USACE regulations state that once a local sponsor has accepted a project, USACE may no longer expend federal funds on construction or improvements. This prohibition does not include repair after a flood. Federally authorized flood control projects, such as the Lake Ponchartrain project, are eligible for 100 percent federal rehabilitation if damaged by a flood.

The local sponsor has a number of responsibilities. In accepting responsibilities for operations, maintenance, repair, and rehabilitation, the local sponsor signs a contract (called Cooperation Agreement) agreeing to meet specific standards of performance. This agreement makes the local sponsor responsible for liability for that levee. For most of the levees surrounding New Orleans, the Louisiana Department of Transportation and Development was the state entity that originally sponsored the construction. After construction, the state turned over control to local sponsors. These local sponsors accepted completed units of the project from 1977 to 1987, depending on when the specific units were completed. The local sponsors are responsible for operation, maintenance, repair, and rehabilitation of the levees when the construction of the project, or a project unit, is complete.

In development of the NOFDS, the Corps of Engineers had the primary responsibilities for development of the concepts, design, and construction (Collins 2005; National Academy of Engineering 2006). Once construction was completed, the operations and maintenance were then turned over to the responsible state and parish agencies. At the federal level, the Corps of Engineers had important interfaces with the executive branch (e.g., Department of Defense and the White House), the legislative branch (Congress), and the judicial branch. Important interfaces also developed with state, parish, and city government agencies, industry, and with the general public. The Industry component is represented by commercial enterprises that are involved throughout the life-cycle of the system including concept development, design, construction, operation, and maintenance. The Public component is represented by national, state, and local individuals and groups that are concerned with and influenced by the NOFDS.

12.4 Extrinsic Factors

Failure of the NOFDS is firmly rooted in *Extrinsic* factors associated with human and organizational performance (Appendix F; Rasmussen 1997; Svedung and Rasmussen 2002; Bea

2006). Causes of the NOFDS failure spanned the full spectrum of organizational failures: cultures, communications, lack of knowledge, use of existing technology, structure and organization, management, leadership, monitoring and control, and mistakes. Mistakes involved breakdowns in perceptions, interpretations, decisions, discrimination, diagnoses, judgments, and actions. In several notable cases, doing things right and doing the right things apparently were surrendered to getting the job done in an expedient way. These observations were summarized by the Select Bipartisan Committee to Investigate the Preparation for and Response to Hurricane Katrina (2006):

Both USACE and the local sponsors have ongoing responsibility to inspect the levees. Annual inspections are done both independently by USACE and jointly with the local sponsor. In addition, federal regulations require local sponsors to ensure that flood control structures are operating as intended and to continuously patrol the structure to ensure no conditions exist that might endanger it. Records reflect that both USACE and the local sponsors kept up with their responsibilities to inspect the levees. According to USACE, in June 2005, it conducted an inspection of the levee system jointly with the state and local sponsors. In addition, GAO reviewed USACE's inspection reports from 2001 to 2004 for all completed project units of the Lake Ponchartrain project. These reports indicated the levees were inspected each year and had received 'acceptable' ratings.

However, both the NSF-funded investigators and USACE officials cited instances where brush and even trees were growing along the 17th Street and London Avenue canals levees, which is not allowed under the established standards for levee protection. Thus, although the records reflect that inspections were conducted and the levees received acceptable ratings, the records appear to be incomplete or inaccurate. In other words, they failed to reflect the tree growth, and of course, neither USACE nor the local sponsor had taken corrective actions to remove the trees.

Complex formal and informal organizations developed that involved a multiplicity of federal, state, parish, city, commercial - industrial, and public enterprises. These organizations had vastly different means, methods, and resources that evolved in different ways at different times. Executive, legislative, and judicial forms of government provided a primary framework for interactions with commercial, industrial, public, and private enterprises. Malfunctions within and between these organizational elements provided the primary element responsible for the failure of the NOFDS (Government Accountability Office 2005, Members Scholars of the Center for Progressive Reform 2005, Select Bipartisan Committee to Investigate the Preparation for and Response to Hurricane Katrina 2006, Townsend 2006, Houck 2006). Ineffective leadership and management were evident before and after failure of the NOFDS. Leonard and Howitt observed (2006):

The leadership failures that contributed to the events we witnessed on the Gulf Coast last August and September began long, long before Katrina came ashore. It literally took centuries to make the mistakes that rolled together to make Katrina such a vast natural and human-made calamity. First, for hundreds of years, people have been constructing and placing large amounts of precious (human lives) and expensive (infrastructure, homes, communities) value in New Orleans and along the Gulf Coast in the known path of severe storms. Second, for decades, we have been living with inadequately designed, built, or maintained

man-made protections (levees, building codes, pumps, and so on), and have pursued policies and interventions that actively contributed to the destruction of the natural buffers (salt marshes, dunes, and other natural barriers) against the hazards created by placing value in harm's way. Third for years - at least since 9/11, but even before that - we have known that we had systems of preparation and response that would prove inadequate against truly large scale disasters. Fourth, in the days and hours before Katrina's landfall, we failed to mobilize as effectively as we might have those systems that we did have in place. And fifth, the days following the impact, we did not execute even the things that we were prepared to do as quickly and smoothly as we should have. How do we not, in the future, find ourselves again with those same regrets? Our work needs to begin with a judicious and honest assessment of threats, followed by investments in prevention and mitigation and by construction of response systems that will be equal to a larger of class of disturbances than we have previously allowed ourselves to contemplate."

In development of the analysis of Extrinsic factors involved in failure of the NOFDS, it is important to recognize that while the Corps of Engineers was primarily responsible for design and construction of the NOFDS and the local state and parish organizations (e.g., Department of Transportation and Development, Levee Boards, Sewerage and Water Boards) were primarily responsible for operations and maintenance, these organizations were subjected to a wide variety of influences and constraints provided by their executive, legislative, judicial and public constituents. The responses of these multiple organizations to provide an adequate NOFDS was clearly lacking. The Senate Committee on Homeland Security and Governmental Affairs report supports this (Leonard and Howitt 2006).

For many years, the Corps of Engineers was severely criticized for delays and cost increases in the Lake Pontchartrain and Vicinity Hurricane Protection Project (Government Accountability Office 1972, 1982, 2005b, 2005c; Carter 2003, 2005a, Carter and Sheikh 2003, Carter et al 2005). Many of these delays and cost increases were reflections of challenges posed by local cost sharing and participation requirements. Local participation and funding requirements introduced additional problems as did interactions with the general public. Additional complexities were added by federal and state legislative, executive, and judicial participation in the developments (lots of 'managers' with different goals, objectives, means, and methods).

At the federal level a long and complex process is required to identify, define, select, and develop projects and secure funding authorizations (Carter and Hughes 2005, Carter 2005d). Historic problems exist with project backlogs, increases in funding requirements, reprogramming actions to manage project funds, and even the fundamental basis for project selection; cost-benefit analyses (Government Accountability Office 1983, 2003, 2005). In short, the Corps must operate in a world that is not of its own making. Outside pressures on the Corps have been negative as well as positive in terms of their effects on performance.

This study indicates that the historic procedures utilized to develop the cost-benefit analyses employed by the Corps of Engineers were and are seriously flawed. These procedures are apparently responsible for some of the seemingly illogical elements in the NOFDS. All costs and all benefits are not incorporated into these analyses (General Accountability Office 2003, Heinzerling and Ackerman 2002). These analyses fail to recognize many important

considerations, uncertainties and projected future developments (Government Accountability Office 2003).

Because of the multitude of recognized deficiencies presently incorporated into traditional Corps cost-benefit analyses, flawed information is provided to policy makers to help them make wise decisions regarding provision of financial resources to develop an adequate NOFDS. Within the executive branch, the Office of Management and Budget has the responsibility to ensure the quality of cost-benefit analyses and resource recommendations, yet the deficiencies were not effectively addressed.

This study indicates that many of the flaws that were introduced into the NOFDS came from flawed decision making regarding provision of financial resources by many organizations at many levels, times, and places. The exceedingly complex and flawed organizational system and its decisions regarding provision of resources that evolved during development of the NOFDS was a primary cause of the failure of the NOFDS. The Corps of Engineers does have and use advanced methods to evaluate costs, benefits, and risks for flood damage reduction studies and dam safety (National Research Council 1983; U.S. Army Corps of Engineers 1996; Powers 2005). Application of these advanced methods was not, however, in evidence in the background available on development of the NOFDS. The substantial body of technology developed to assist risk management decision making (e.g., Fischhoff et al. 1981; Wenk 1989; Shapira 1995; Molak 1997; Kammen and Hassenzahl 1999; Spouge 1999; Moteff 2004; Jordan 2005; see Appendix H) should be further developed, codified and applied by the Corps to assist policy makers in making decisions regarding development and maintenance of levees and flood protection systems.

For many years, the Corps of Engineers has been subjected to extreme pressures at the federal and state levels to do more with less (Government Accountability Office 1997; Office of Management and Budget 2006); to do their projects better, faster, and cheaper; and improve project management (planning, organizing, leading, controlling). The organization's attempts to respond to all of these frequently conflicting pressures has introduced organizational turbulence and diversion of attention and resources that continues the present time. The Corps of Engineers developed a plan to re-engineer itself (U.S. Army Corps of Engineers, 2003b). However, it is clearly struggling with all of its constraints to achieve key elements in this plan (Office of Management and Budget 2006). Our study indicates that as in the case of NASA (Appendix F) technical and engineering superiority and oversight was compromised in attempts to respond to all of these constraints and pressures; especially those pressures for increased efficiency and decreased costs. Adequate quality and reliability in the constructed works has suffered and will continue to suffer until these challenges are successfully addressed.

Evidently the organizations responsible for the various parts of the life-cycle of the NOFDS did not have effective process auditing procedures (Knoll 1986). They did not have incentive systems that discouraged excessive and inordinate risk taking that could lead to less than desirable quality and reliability. Quality standards did not meet or exceed the referent standards required for a high quality and reliable NOFDS. These organizations did not correctly assess the risks associated with given problems or situations; apparently, situational awareness was frequently lost. These organizations lacked strong command and control systems as evidenced with appropriate rules and procedures, effective selection and training of personnel, decisions being made in the right ways at the right times by the right people, effective redundancy (robustness) to create tolerance to organizational defects, and maintenance of situational awareness for appropriate action. In general, these organizations performed as Low

Reliability Organizations (Weick and Sutcliffe 2001). Effective leadership and management was lacking (Townsend 2006; Collins 2005; Government Accountability Office 2006). Such organizational malfunctions were summarized by Irons (2005):

The evidence indicates the U.S. Army Corps of Engineers knew about the threat of breaches, as opposed to overtopping, since the early 1980s. Moreover, all concerned agencies, including those at the local, state, and federal levels, knew about the threat of overtopping and consequent flooding in even a Category 3 hurricane.

Basic flaws in the design of the levee protection system were first recognized over two decades ago, before the wetlands were so diminished. An outside contractor, Eustis engineering, was the first to express concerns about the levee vulnerability to breaching in the early 1980s. In 1981, the New Orleans Sewerage & Water Board developed a plan to improve street drainage by dredging the 17th Street Canal. The Corps of Engineers issued permits to do the dredging in 1984 and 1992, though the Corps was not a partner in the Project. Eustis Engineering contracted to do a design study for Modjeski and Masters, the consulting engineers on the project, and performed soil investigations on a section of the 17th Street Canal from south of the Veterans Memorial Boulevard bridges to just north of those structures. They found that 'the planned improvements to deepen and enlarge the canal may remove the seal that has apparently developed on the bottom and side slopes, thereby allowing a buildup of such pressures in the sand stratum.' Eustis' concerns about a 'blow-out', or breach, of the levee were strong enough that the company recommended test dredging before the final design.

...The most puzzling point about the dredging project is that the Corps of Engineers planned to follow the project by raising the floodwall from 10 feet to 14.5 feet. It is unclear whether the Corps paid attention to the contractor's concerns since most of the documents related to the work remain unavailable to the public. Although the Corps of Engineers was not a direct partner in the dredging, it was aware of the work and knew it would have an impact on its later project. Indeed, contractors working for the Corps on the later project raised their own concerns about the soil and foundations of the levee.

Reports indicate that key sections of the levee system's soil and foundation, particularly the floodwall on the 17th Street Canal where much of the serious flooding occurred, posed serious problems for the contractors involved. Court papers from 1998 show that Pittman Construction indicated to the Corps of Engineers as early 1993 that the soil and foundation for the walls were 'not of sufficient strength, rigidity and stability' to build on. The construction company claimed that the Corps of Engineers did not provide it with complete soil data when it developed a bid on the levee project.

...Engineers now say the difficulties Pittman Construction faced were early warning signs that the Corps of Engineers ignored. The Corps of Engineers officially disputed the points made by Pittman Construction regarding the soil condition, though it now seems clear that the crucial breaches in New Orleans occurred in levees where the floodwall foundations were not as deep as the canals and that the Corps of Engineers was aware of the issue.... Would an organization

with processes in place to support ongoing learning, and surprise-avoidance, fail to recognize the legitimacy of the contractor's point rather than argue about purely budgetary issues related to the contract?

Principal knowledge related malfunctions centered on inappropriate use of existing technology (“unknown knowables”) and inadequate measures to disclose unknowns throughout the life-cycle of the NOFDS. Examples include the subsidence and settlements of critical flood protection elements in the NOFDS including those of the floodwalls along the drainage canals (which are now known to be about two feet below intended elevations), the Industrial Canal floodwalls (about three feet below intended elevations), and the levee elevations along the Mississippi River Gulf Outlet (MR-GO) that front the south side of Lake Borgne (about two to three feet below intended elevations) (Interagency Performance Evaluation Task Force, 2006a, 2006b). Concerns regarding settlements and subsidence were expressed early in the development of the NOFDS, but apparently no effective action was taken to quantify the regional subsidence and settlements and to make appropriate adjustments to the NOFDS. Even though information was developed by the National Geodetic Survey that the reference benchmarks being used as controls in construction of the NOFDS were in excess of one foot low, the decision was made in August 1985 to use the benchmarks “current at the time of construction of the first increment of the project” (1965) (Chatry 1985). The report of the Senate Committee on Homeland Security and Governmental Affairs observed (2006):

In Designing, constructing and maintaining the hurricane-protection system the Corps did not adequately address: (a) the effects of local and regional subsidence of land upon which the protection system was built; and (b) then-current information about the threat posed by storm surges and hurricanes in the region.

Another important example of knowledge development and utilization malfunctions is that of the overtopping and breaching of the levees and flood control structures along the MR-GO. Of particular importance is the stretch of levee that defends St. Bernard Parish between the bayou Bienvenue and bayou Dupre flood control structures. This stretch of levee was badly damaged during hurricane Katrina as were sections where the levee joined the flood control structures (Seed et al., 2005). The current work of the ILIT indicates that the sections adjacent to the flood control structures breached where the construction had covered the original bayou channels. The design of the junctions between the flood control structures and the earth levee were not sufficient to withstand the surge and wave action developed during hurricane Katrina. In a similar manner, the levees between bayous Dupre and Bienvenue were not able to withstand the waves and surge that developed across lake Borgne; they were severely breached and massively eroded. The ILIT indicates that the wave and surge velocities that preceded the arrival of the peak surge likely were initiating breaching before these levees were overtopped so that when the levees were overtopped, the breaches could be readily expanded and allow large volumes of water to enter the protected areas in St. Bernard parish. In contrast, the performance of the levee north of bayou Bienvenue to its intersection with the GIWW was markedly different. Our studies indicate that this performance resulted from a combination of factors that included superior soils used in construction of this stretch of levee (highly erosion resistant) and natural protection (water velocity reduction) afforded by adjacent wetlands on the outboard side of the levee.

The MR-GO is a 76-mile long navigation channel connecting the Gulf of Mexico to the Port of New Orleans Inner Harbor Navigation Canal via the GIWW. The channel bisects the marshes of lower St. Bernard Parish and the shallow waters of Chandeleur Sound. Construction

of this shipping channel/canal was authorized by Congress in 1956. Its construction was started in 1958 and completed in 1965. Many people contend that the MR-GO played a prominent role developing the flooding of St. Bernard parish and East New Orleans during hurricane Betsy (1965). Before, during, and after construction of the MR-GO (48 years) many concerns were expressed regarding the effects of this canal on the adjacent wetlands and on its potential focusing effect on storm surge propagation into the IHNC. Originally conceived as a way to get deep draft ships to the Port of New Orleans facilities in the IHNC, it failed to realize its commercial justification because of changes in ships (deeper drafts) and because of the need for almost continuous dredging to keep the channel open. The channel also allowed the highly saline waters from the Gulf of Mexico to intrude into the adjacent fresh and brackish water wetlands and marshes, destroying many in the process (estimated more than 20,000 acres of marsh have been destroyed). In 1988, the St. Bernard Parish Council unanimously adopted a resolution to close MR-GO because it constituted a threat to public health and safety. In October 2004, the Louisiana Legislature passed a resolution urging closure of the MR-GO and immediate implementation of remedial measures to address the risk posed by the MR-GO.

Available information and soil sampling conducted during this study indicates that the levees between bayous Bievenue and Dupre originally were constructed from dredge spoil deposited during the construction of the MR-GO (A. Theis, personal communication, January 2006) (see Chapters 6 and 9). The USACE's own design documentation states that the materials used are potentially susceptible to erosion (USACE, DM-____, 19__). Additional construction was proposed to increase the height of the levee at the time of hurricane Katrina. While these materials were highly susceptible to scour and erosion, the ILIT study has failed to discover documentation of plans or proposals for armoring this levee prior to hurricane Katrina.

Given the recognized degradation of the protection afforded by wetlands (Hallowell 2001), recognition of the erodability of the levee soils, the lack of provision of protection for the levee soils, recognized deficiencies in the design criteria, the continued challenges of keeping these levees at their authorized elevations (significant subsidence and compression), and the repeated expressions of concerns for the adequacy of these protective works, the performance of this part of the NOFDS was a "predictable surprise." The Member Scholars of the Center for Progressive Reform (2005) arrived at similar conclusions.

Rejection and misuse of technology are evident in the history of the NOFDS. Interactive risk assessment and management approaches (e.g., Quality Assurance and Quality Control) (Knoll 1986; Loosemore 2000) to help detect, analyze, and correct knowledge related challenges apparently failed for a wide variety of reasons including excessive authority gradients, low task and situational awareness, excessive professional courtesy, cultural-societal morays, excessive beliefs, deficiencies in communications, and deficiencies in resource and task management. Irons observed (2005):

The U.S. Army Corps of Engineers is historically an insular agency, known for doing things its own way. It is not possible to say whether surprise-avoidance processes are in place at the Corps of Engineers, until the public receives more access to internal documents. The failure of Corps' staff to recognize and prioritize the challenges of levee upgrades and receding wetlands to the city of New Orleans, and surrounding areas, strongly suggests that surprise-conductive processes characterize its organization. The Corps' organization has over the past few decades outsourced more work, lost many engineers to private industry, and consequently suffered a diminished capacity to attract top-notch engineers.

New Orleans had dodged the bullet many times, with the major force of hurricanes skirting around the area. Nevertheless, most people with a reason to know about it were aware that a Category 3 hurricane posed a severe threat to the New Orleans' levee protection system, and a Category 5 hitting land as a Category 4, as with Katrina, posed a catastrophic threat.

The occurrence of a hurricane like Katrina was not unexpected in New Orleans; neither were the complications faced in the aftermath of the storm. Given this understanding, and the neglect in preparing for a hurricane like Katrina, as well as the ineffective response preparations, it seems reasonable to assert that Katrina as well as its aftermath was a predictable surprise. The threats posed by the hurricane, and the likely aftermath, were well known and unsurprising to most who thought about the hurricane threat to New Orleans. Unfortunately, much of the local, state, and federal leadership, especially the U.S. Army Corps of Engineers, appears to have remained complacent about preparing the levees for a catastrophic hurricane.

All of these Extrinsic factors represent corporate failures in making decisions that involved all components of the TDS, including the public. The right things were traded-off for the wrong things at the wrong times and in the wrong ways. The failure of the NOFDS has all of the same ingredients found in previous catastrophic failures and accidents (Appendix F). It involved many different people and organizations developing a wide variety of malfunctions (e.g., decisions) over a long period of time (40 years). While a majority of these malfunctions were embedded during the concept and design phases, early warnings that indicated 'all was not well' as the NOFDS progressively developed were not detected, analyzed, and corrected. When hurricane Katrina finally tested the flawed, defective, and deficient NOFDS, it failed catastrophically producing the single most catastrophic failure of an engineered system in the history of the United States.

12.5 Intrinsic Factors

Intrinsic factors representing natural variability and analytical modeling uncertainties also played roles in the failure of the NOFDS (Vick 2002; Bea 2006). There are fundamental flaws in the basic criteria and guidelines that were used to design the NOFDS. These flaws include engineering elements that address:

- Design *demands* for the elements of the NOFDS; including the Standard Project Hurricane (SPH) conditions (surge heights in the NOFDS, frequency of occurrence, and lack of explicit recognition of the likely effects of more intense hurricanes) (Select Bipartisan Committee to Investigate the Preparation for and Response to Hurricane Katrina 2006, ASCE 2006a). Even though studies after 1972 indicated the need for increases in the design flood protection elevations due to greater surge and wave heights, these were not reflected in revised design guidelines (Brouwer 2003; Carter 2005a, 2005b).
- Design *capacities* for the elements of the NOFDS; including engineering guidelines used to design and construct the levees and floodwalls (e.g., analyses of levee stability, levee stability factors of safety, analyses of floodwall/sheetpile stability, deformation and stresses, floodwall design factors of safety, provision for deformations in the floodwalls during surge loading, provisions for robustness - defect and damage tolerance and fail-safe performance, and provisions for subsidence) (Select Bipartisan Committee to Investigate the Preparation

for and Response to Hurricane Katrina 2006; Vartabedian and Braun 2006; Irons 2005; ASCE 2006b).

- Configuration of the elements that comprise the NOFDS as an *integrated flood defense system* (Seed et al. 2005, ASCE 2006a). Many failures of the NOFDS occurred at a variety of types of interfaces in the physical elements such as interfaces between earth levees and concrete and steel flood protection elements and between the flood control structures and pump station structures (Carter 2005a). Flood discharge pumps were not sufficiently protected from backflow and exacerbated flooding. Many vulnerabilities were found at transitions and interfaces between flood protection elements and/or where other infrastructure elements were involved (Seed et al. 2005). The NOFDS was not an integrated, coherent system; rather “*it is a jointed series of individual pieces conceived and constructed piecemeal*” (ASCE 2006a).

12.5.1 Standard Project Hurricane

The heart of most Corps of Engineers hurricane protection projects since the 1960s has been the Standard project Hurricane, or SPH (Carter 2005a, Government Accountability Office 2005a, 2005b). The SPH was developed by the National Weather Service and the Corps of Engineers at the request of Congress in the 1950s “to provide generalized hurricane specifications that are consistent geographically and meteorologically for use in planning, evaluating and establishing hurricane design criteria for hurricane protection works” (Graham and Nunn 1959). Attempts to describe the SPH in terms of Categories has lead to confusion because the SPH preceded development of the Saffir-Simpson hurricane scale (Categories). Depending on what characteristic of a hurricane is referenced, the SPH for the NOFDS can vary from a Category 2 to a Category 4 storm.

A primary goal of the SPH was to compare hurricane protection standards from region to region (Perdikis 1967). This standardized approach led to disparities within a particular region. The SPH model excluded storms that were deemed to be inordinately severe. For example, the 1979 revision of the SPH removed two particularly severe storms from the data base; hurricane Camille of 1969 and the Labor Day Hurricane of 1935. Experience shows that excluding outlier data is not appropriate in the context of dealing with extreme hazards. In addition, a higher standard of protection was specified for facilities and areas “where high winds, waves and storm surge could pose a threat to the public health and safety from a hurricane-induced accident at a nuclear power plant” (Schwert et al 1979). This shows that the SPH criteria includes an implicit cost-benefit assessment. This implicit assessment prevents policymakers (and the public they represent) from determining whether an extreme event is worth guarding against by excluding the possibility that such an event will or can occur. The following quotations indicate the interpretations that developed through the history of development of the NOFDS regarding what the system’s SPH represented.

- “The Standard Project Hurricane wind field and parameters represent a ‘standard’ against which the degree of protection finally selected for a hurricane protection project may be judged and compared with protection provided at projects in other localities.” (Graham and Nunn 1959).
- “The project is designed to protect against the Standard Project Hurricane moving on the most critical track. Only a combination of hydrologic and

meteorological circumstances anomalous to the region could produce higher stages. The probability of such a combination of occurring is, for all practical purposes, nil.” (U.S. Army Corps of Engineers 1974).

- “The SPH is a steady state hurricane having a severe combination of values of meteorological parameters that will give high sustained wind speeds reasonably characteristic of a specified coastal location. By reasonably characteristic is meant that only a few hurricanes of record over a large region have had more extreme values of the meteorological parameters.” (National Weather Service 1979).
- “The SPH was expected to have a frequency of occurrence of once in about 200 years, and represented the most severe combination of meteorological conditions considered reasonably characteristic for the region.” (Government Accountability Office 2005).

As can be seen, over time the SPH went from being a general indicator of threat levels to a guarantee of safety. The methods used to define the SPH were buried, along with their potential flaws and questionable assumptions. Because it became the “gold standard” of flood system performance, the SPH served to prevent up-to-date reanalysis of the true risks of catastrophic flooding of the NOFDS.

Recent work indicates that the probability that a hurricane will pass within 75 miles of New Orleans in any given year is about 12.5 percent, or about once every eight years (URS 2005). The likelihood of a major hurricane (Category 3 and above) are about 3.2 percent per year, or about once every 30 years. These projections do not account for the current period of intensified hurricane activity (Klotzbach and Gray 2006). Thus, the history of development and evolution of the SPH did not provide an adequate basis to understand the risks associated with catastrophic flooding of the NOFDS. The report of the Senate Committee on Homeland Security and Governmental Affairs observed (2006):

For several years, the Corps has inaccurately represented to state and local officials and to the public the level of protection that the hurricane system provided. The Corps claimed the system protected against a fast-moving Category 3 storm even though: (a) there was no adequate study or documentation to support this claim; and (b) information known to or provided to the Corps demonstrated that the claim was not accurate.

Some industries (e.g. offshore engineering) that must deal with hurricane related hazards have developed specific design conditions (e.g., wave or surge height) and associated forces based on specified annual return periods (e.g., 100 years) (Bea 1990, 1998, 2001). These design conditions are chosen based on their potential *effects* (e.g., forces, water surface elevations) *on the structures to be designed*. The design conditions and prescribed forces are supplemented with safety factors (e.g., 2 to 4) that help assure that the resulting system can perform acceptably in much more intense conditions (frequently identified as Ultimate Limit State conditions) (Bea 1990). For important industrial facilities, these Ultimate Limit State conditions have return periods in the range of 1,000 to 10,000 or more years (Vick 2002; Baecher and Christian 2003, U.S. Army Corps of Engineers 1999, Tekie and Ellingwood 2003). For example, typical modern offshore structures in the Gulf of Mexico that are evacuated in advance of hurricanes are designed to be able to resist forces from hurricanes that have return periods of more than 1,000 to 5,000 years (Bea 1996). Structures that cannot be evacuated are designed to be able to resist forces from hurricanes and storms that have return periods in the range of 5,000 to 10,000 years

(Spouge 1999). In a similar vein, the Dutch currently provide protection against flooding of the Netherlands for events that represent the worst storm that could be expected to affect the area with return periods in the range of 1,000 to 10,000 years (Versteeg 1988, Netherlands Water Partnership 2005).

The SPH evolved to represent the most severe storm the government should guard against when designing hurricane protection projects. The SPH came to represent not only a method for comparative assessment of storm risks between geographic areas, but also a design standard that carried its own assurance of adequate reliability. For a variety of reasons, the concept of storms much more intense than the SPH was not allowed to explicitly enter the engineering process, even though the development of the SPH also involved a Probable Maximum Hurricane (PMH) (National Weather Service 1979):

The PMH is a hypothetical steady state hurricane having a combination of values of meteorological parameters that will give the highest sustained wind speed that can probably occur at a specified coastal location. One of several possible uses of the values of meteorological parameters is to compute maximum storm surge at coastal points when the hurricane approaches along the most critical track [authors' emphasis].

Thus, it was clearly recognized that the SPH did not represent a maximum set of conditions for design against hurricane conditions. It is also clear that general public was not informed about the flooding risks that the selection of the SPH as a basis for design implied. In many cases, even though very inexpensive defenses could have been provided for the potential for hurricane surges exceeding those of the SPH (e.g., splash-pads behind I-walls and other similar floodwalls sensitive to overtopping erosion), these defenses were not provided.

Another important element of the SPH was that it was not revised as knowledge improved after the 1960s. Authorization constraints and engineering restraints were provided to us as an explanation for this (bureaucratic engineering). Tremendous strides in the meteorology and oceanography of hurricanes were made during the 1970's, and these improvements in technology continue to evolve to the present time (Simpson 2003, Interagency Performance Evaluation Task Force 2006a, 2006b). However, the SPH remains essentially the same as it was when it was initially defined in the 1950s and early 1960s. The natural variability in hurricane conditions, and the ability of these conditions to exceed the design norms of the SPH, and how these norms were translated in design resulted in many of the failures observed in the NOFDS in the wake of hurricanes Katrina and Rita.

12.5.2 Failure Modes and Safety Factors

A primary obligation of an engineer is to anticipate failure modes in the element, component, or system being engineered and then provide measures to prevent those failure modes from developing or from developing catastrophic results (Petroski 1985, 1994; Harr 1987; Wenk 1989, 1995, 1998; Appendix H). This obligation requires two primary things: (1) anticipation of possible failure modes, and (2) provision of defenses in depth to prevent and/or mitigate those failure modes.

The design demand for a particular component in the NOFDS, when combined with a prescribed safety factor and associated analytical models and procedures, determines the Ultimate Limit Strength of that element. When combined with an assessment of the intrinsic uncertainties (natural, model, parametric, state), the ratio of the Ultimate Limit Strength to the

design demand (factor of safety) reflects the reliability (or probability of failure) associated with that component (Bea 1990; Kulhawy 1996; Duncan 2000; Whitman 2000; Vick 2002; Christian 2004; Lacasse 2004).

For example, a factor of safety of 1.3 was specified by the Corps of Engineers as the minimum acceptable safety factor for drainage canal levee lateral stability for the “transient” loading conditions represented by hurricane-induced storm surge and waves (U.S. Army Corps of Engineers 1988, 1989, 1990, 2000). Our examination of the basis for this factor of safety indicates that it was developed in the 1950s for levees used primarily to defend sparsely populated agricultural areas (Wolff, 2005). This factor of safety is embodied in current Corps of Engineers guidelines for the design of levees and assessment of slope stability (U.S. Army Corps of Engineers 2000, 2003). In the case of the drainage canal levees and those along the IHNC, and the floodwalls constructed on and in these levees, the design demand was determined by the total lateral force represented by the canal water level determined on the basis of the SPH (U.S. Army Corps of Engineers 1994).

In the 1990s the Corps of Engineers developed very advanced analytical methods to assess the reliability of important flood control components such as levees and dams (U.S. Army Corps of Engineers 1996, 1999, National Research Council 1983). These methods were validated with field and laboratory test data and field performance data (U.S. Army Corps of Engineers 1999; Wolff 1999; Duncan 2000). Application of these methods entailed an assessment of the inherent uncertainties for different failure modes and identification of target reliabilities for these modes (Wolff, 1999). Analytical methods were developed for both elements and assemblies of elements that represented flood control components and systems. The issues associated with ‘target’ or acceptable reliabilities were also addressed. These methods were used to define reliability-based design and maintenance factors of safety. Application of these methods for important flood protection facilities defending highly populated areas and the Corps of Engineers levee stability analysis procedures indicated the need for factors of safety that substantially exceeded those actually used for the levees and associated floodwalls that defended the NOFDS drainage canals. A need for “Factors of safety” of 2 to 3 and greater were indicated for very important facilities (annual target Safety Indices in the range of 3 to 4). Similar safety factors were identified by other investigators for similar facilities (Bea 1990, Duncan 2000, Whitman 2000, Vick 2002, Christian 2004, Lacasse 2004). Apparently a *technology lag* (breakdown in technology transfer) or rejection of technology (Sowers 1993) developed and persisted in the design guidelines used for levees and floodwalls in the NOFDS. As a result, standard design Factors of Safety were inadequate, and overall system reliability was compromised as a direct result.

Following Hurricane Katrina a similar technology lag was identified as one of the causes of the failure at the 17th Street canal. Both the ILIT (Seed et al. 2005) and the Corps of Engineers Interagency Performance Evaluation Task Force analyses (Interagency Performance Evaluation Task Force 2006b) of this failure concluded that a failure mode developed that was not recognized by the designers. This finding led to the official contention that this was a “design failure.” *The information developed by the ILIT clearly indicates that this failure was a result, not a cause.*

This failure mode involved lateral deflection of the concrete floodwall and the sheet piles that supported that floodwall. This deflection resulted in separation between the stiff supporting sheet piling and the soft soil of the levee on the outboard side (flood side) of the wall. Water was then able to enter the gap and exert additional lateral forces against the lower portions of the

sheetpiles and thus on the remaining (inboard) ‘half’ of the levee (and floodwall. Now the levee only had about ‘half’ of its width and mass able to transmit the lateral forces to the underlying soils. This combination resulted in lowering the lateral resistance with a commensurate lowering of the factor of safety.

This development was incorrectly reported as “unforeseen and unforeseeable” by the Interagency Performance Evaluation Task Force on March 10, 2006 (Marshall 2006; Seed and Bea 2006). In 1985, the New Orleans district of the Corps of Engineers conducted a full scale instrumented lateral load test of a 200-foot long sheet pile / flood wall in the Atachafalaya basin [the E99 sheetpile test] (U.S. Army Corps of Engineers 1988b). This particular location (south of Morgan City, Louisiana) was chosen because of the close correlation of the soil conditions in the New Orleans area with those at the test location. “The foundation soils are relatively poor, consisting of soft, highly plastic clays, and would be representative of near worst case conditions in the NOD (New Orleans District).” (U.S. Army Corps of Engineers 1988b).

Test data from the highly instrumented sheet pile wall and adjacent supporting soils indicated a gapping behavior (separation of the sheet piles from the soils). The test was designed to take an eight foot height of water (above the supporting ground level) with a factor of safety of 1.25. But, the wall was already in a failure condition (increasing lateral displacements with no increase in loading) when the water level reached only 8 feet instead of the calculated 10 feet. Strain gage readings on the sheet piles indicated that they were well below the steel yield point, thus the yielding had to have been developing in the supporting soils. Two very important pieces of information developed by the E-99 sheet pile tests were that there was potential soil separation from the sheet piles (allowing water to penetrate below the ground surface between the piles and the soils) and that the calculated safety factor was not reached (it was over-estimated due to unanticipated deformations in the soils).

Additional reports and professional papers further developed the experimental information and advanced analytical models that could be used to help capture such behavior (U.S. Army Corps of Engineers Waterways Experiment Station 1989b). Later developments in this work confirmed the gapping between the sheetpiles and the outboard side of the levee embankment, and were published in USACE reports and were eventually reported in the open professional engineering literature by Oner, Dawkins and Mosher (1997):

As the water level rises, the increased loading may produce separation of the soil from the pile on the flooded side (i.e., a “tension crack” develops behind the wall). Intrusion of free water into the tension crack produces additional hydrostatic pressures on the wall side of the crack and equal and opposite pressures on the soil side of the crack. Thus part of the loading is a function of system deformations.

These developments in technology inexplicably were *not* reflected in the design guidelines used (U.S. Army Corps of Engineers 1988a, 1989a, 1990). We also found no evidence that questions regarding the adequacy of the design were raised after the design and construction were completed. Loss of corporate memory, breakdowns in technology transfer, and abilities to keep the design guidelines current with existing knowledge seemed to background these developments.

A second suspect element in the development of the failure at the 17th Sreet Canal regarded characterizations of the soils that supported the earth levee and sheet piling in the vicinity of the 17th Street canal breach. The processes used at the time of design to analyze the

soil types and engineering characteristics did not capture the unique characteristics of the soils. Soil strengths based on samples from beneath the crest of the levee, with higher strengths resulting from higher overburden loads and thus compression of these soils to a denser state, were inappropriately used to characterize the strengths of the soils at and beyond the toes of the levees (where lower overburden loads resulted in lower strengths). In addition, the spatial averaging process (vertical and lateral) did not capture the unique soil characteristics in the vicinity. Soils in Southern Louisiana and other parts of the Gulf Coast have very complex histories due to past floods, hurricanes, the rise and fall of sea level, changes in vegetation, and other events. Far from being uniform, they contain complicated and rapidly varying strata of different materials with very different characteristics.

In 1964 - 1965 the Corps ran a full scale levee test in the Atachafalaya basin in which advanced studies were conducted regarding characterizations of the soil strengths and performance – stability characteristics of the levee (U.S. Army Corps of Engineers 1968; Kaufman and Weaver 1967). The levee test sections were thoroughly instrumented and their performance monitored during and after construction. Various analytical methods were used to evaluate the usefulness and reliability of the various methods. These developments clearly indicated the need to understand the geologic soil depositional processes and the associated variations in soil strengths (horizontal and vertical) in order to understand the performance and stability characteristics of levees. The importance of local soil conditions to performance of the levee was clearly pointed out. The tremendous importance of overburden loads on soil strengths was a major focus of this work; and led to award-winning advancement of the Stress History and Normalized Engineering Performance (SHANSEP) framework for evaluation and modeling of the strengths of these types of soils. Additional reports and professional papers were published that resulted in significant advances to the engineering knowledge (Duncan 1970, Ladd et al. 1972; Edgers et al. 1973; Foott and Ladd 1973, 1977). None of these vital principals, however, were subsequently incorporated in the design and analysis of the 17th Street canal levees and floodwalls.

In-depth background and understanding of the geologic and depositional environment and history of vital importance to understanding the characteristics of the Mississippi Basin soils were developed in the 1950s and 1960s (Fisk et al. 1952; Kolb and Van Lopek 1958; Krinitzsky and Smith 1969) and the Corps of Engineers led in the development of this background. Of particular importance was recognition that the marsh and swamp deposits were “treacherous” and highly variable. It was repeatedly pointed out that “careful and detailed characterization of the soil properties was required.” Further, the studies cited above led to the recognition that the methods based on traditional Corps of Engineers soil characterization and stability analyses gave factors of safety that were unconservative (too large); (Foot and Ladd 1977). As in the first instance, these developments in technology inexplicably were not reflected in the design guidelines and practices that were used in the actual design studies.

The safety factors used in design were not sufficient to accommodate the uncertainties inherent in the design procedures and processes and inherent in the environment in which the facility would exist. Important failure modes in the components were not recognized. When the system was tested, it failed because of a confluence of intrinsic and extrinsic uncertainties. *This was not a design failure; this was a failure on the part of the organizations responsible for the design and construction of the flood defense works to effectively use proven technology.*

12.6 Life-Cycle Development of Flaws

Sources of flaws in the NOFDS developed during the life-cycle of the system starting with its concept (e.g., SPH), then during design (e.g., I-wall configurations, strength and stability guidelines, factors of safety), construction (e.g., normalized reports of excavation and forming instabilities and seepage from canals) and operation (e.g., persistent reports of leakage from canals and signs of ground instability), and finally during the maintenance (e.g., in-ground construction, vegetation growth on and adjacent to levee toes) phases (Select Bipartisan Committee to Investigate the Preparation for and Response to Hurricane Katrina 2006, Irons 2005). Similar life-cycle flaws were developed and propagated in the levee and flood protection structures adjacent to the MR-GO. *Important flaws in the NOFDS were embedded in every stage of the life-cycle.* In many cases, these flaws were allowed to propagate and magnify. Early warning signs were ignored or were ineffectively addressed. NOFDS component interface flaws that developed throughout the life-cycle of the NOFDS were particularly evident.

When the NOFDS was challenged by hurricane Katrina, these flaws became evident. Had these flaws not been present, it is likely that hurricane Katrina would not have developed into a major catastrophe.

Design challenges not successfully addressed were traced to fundamental flaws that became embedded in engineering design procedures and how these procedures were used. Tests were performed and the results not properly utilized, and in several key cases, not utilized at all (Seed and Bea 2006). Even though procedures for other similar facilities (e.g., dams, coastal and offshore structures) existed and were highly developed, the design (also construction, operation, inspection, maintenance, and repair) technology was not integrated into the design of the NOFDS (rejection or misuse). In addition to flaws previously discussed, the design procedures focused on individual components, with insufficient treatment given to the concepts of integrated system performance, defenses in depth, and robustness (damage and defect tolerance). The Member Scholars of the Center for Progressive Reform arrived at similar conclusions in their report titled *An Unnatural Disaster: The Aftermath of Hurricane Katrina* (2005).

12.7 Findings - Looking Back

Failure of the NOFD was not caused by an overwhelming extreme natural event (hurricane wind, waves, currents, surge). While portions of the NOFDS were overtopped by hurricane Katrina's surge and waves, our studies indicate that the majority of the flooding came from unanticipated and unintended breaches in the levees (many adjacent to other structures), failures in the floodwalls, and water entering through gaps (floodgates not in place) or low spots in the NOFDS. The roots of these unanticipated and unintended developments were firmly embedded in Technology Delivery System flaws and malfunctions; failures of organizations - institutions and their resource allocation processes.

ILIT identified eight categories of technology delivery system (TDS) malfunctions that played primary roles in the failure of the NOFDS. Additional background on each of these TDS malfunctions is provided in Appendices F and H.

Failures of foresight: Catastrophic flooding of the greater New Orleans area due to surge from an intense hurricane had been predicted for several decades (Townsend 2006). The consequences observed in the wake of hurricane Katrina were also predicted (Members Scholars of the Center for Progressive Reform 2005). The hazards associated with the NOFDS were not

adequately recognized, defensive measures were not identified and prioritized, and effective action was not mobilized to effectively deal with these hazards (Irons 2005; Senate Committee on Homeland Security and Governmental Affairs 2006).

Failures of organization: The roots of the failure of the NOFDS are firmly embedded in flawed organizational - institutional systems (Select Bipartisan Committee to Investigate the Preparation for and Response to Hurricane Katrina 2006). The organizational - institutional systems lacked centralized and focused responsibility and authority for providing adequate flood protection (Government Accountability Office 2005a, 2005b; Carter 2005a, 2005b; ASCE 2006a; Senate Committee on Homeland Security and Governmental Affairs 2006). Dramatic and pervasive failures in management existed, exemplified by ineffective and inefficient planning, organizing, leading, and controlling to achieve desirable quality and reliability in the NOFDS (Houck 2006, Braun and Vartabedian 2005). There were extensive and persistent failures to demonstrate initiative, imagination, leadership, cooperation, and management (Leonard and Howitt 2006).

Failures of funding: The failure of the NOFDS resulted in part from inadequate provision of resources based primarily on recommendations provided by the Corps followed by failure of the federal and state governments to fund badly needed improvements once limitations were recognized (Members Scholars of the Center for Progressive Reform 2005; Houck 2006; Braun and Vartabedian 2005). In several instances, State agencies pressured for 'lower cost' solutions not realizing that these solutions would result in lowering the overall quality and reliability of the NOFDS (Members Scholars of the Center for Progressive Reform 2005). Important deficiencies existed in the cost-benefit analyses used to justify the levels of protection and their continued improvement as knowledge and technology advanced (Government Accountability Office 2003, 2005; Heinzerling and Ackerman 2002).

Failures of diligence: Forty years after the devastating flooding caused by hurricane Betsy, the flood protection system authorized in 1965 and based on the Standard Project Hurricane (SPH) was still not completed (Government Accountability Office 2005a, 2005b). The concept and application of the SPH was recognized to be seriously flawed, yet no adjustments were made to the system before Katrina struck (Select Bipartisan Committee to Investigate Preparation for and Response to Hurricane Katrina 2006). Early warning signs of deficiencies and flaws persisted throughout development of the different components that comprised the NOFDS and these signs were not adequately evaluated and acted upon (Houck 2006; Carter 2005a, 2005b).

Failures of trade-offs: A history of flawed decisions and trade-offs proved to be fatal to the ability of the system to perform adequately (Carter 2005a, 2005b). Compromises in the ability of this system to perform adequately started with the decisions regarding the fundamental design criteria for the development of the system, and were propagated through time as alternatives for the system were evaluated and engineered (Houck 2006). Design, construction, operation, and maintenance of the system in a piecemeal fashion allowed the introduction of additional flaws and defects (Collins and Lieberman 2005). Efficiency was traded for quality, reliability, and effectiveness. Superiority in provision of an adequate NOFDS was traded for mediocrity and getting along (Collins 2005; Senate Committee on Homeland Security and Governmental Affairs 2006).

Failures of management: Requirements imposed on the Corps of Engineers by Congress, the White House, State and local agencies, and the general public have changed

dramatically during the past three decades. Defense, re-construction, maintenance, waste disposal, recreational, emergency response, and ecological restoration have served to divert attention from flood control (Office of Management and Budget 2006, Vartabedian and Braun 2006). Public and Congressional pressures to reduce backlogs of approved projects, improve project and organizational efficiency (downsizing, outsourcing), address environmental impacts and develop appropriations for projects have served to divert attention from engineering quality and flood control reliability (Carter and Sheikh 2003). Engineering technology leadership, competency, expertise, research, and development capabilities appear to have been sacrificed for improvements in project planning and controlling (Office of Management and Budget 2006; Senate Committee on Homeland Security and Governmental Affairs 2006).

Failures of synthesis: While individual parts of a complex system can be adequate, when these parts are joined together to form an interactive - interdependent - adaptive system, unforeseen failure modes can be expected to develop (Rasmussen 1997; Bea 2000). These unforeseen, but foreseeable, failure modes developed in the NOFDS during hurricane Katrina. It is evident that insufficient attention was given to creation of an integrated series of components to provide a reliable NOFDS (ASCE 2006a). Synthesis was subverted to decomposition. As a result, many failures developed at interfaces or 'joints' in the NOFDS (Committee on New Orleans Regional Hurricane Protection Projects 2006; Seed et al. 2005).

Failures of risk assessment and management: The risks (likelihoods and consequences) associated with hurricane surge and wave induced flooding were seriously underestimated (Carter 2005a, 2005b). There was inadequate recognition of the primary contributors to the likelihoods and consequences of catastrophic flooding. Sufficient defensive measures to counteract and mitigate these uncertainties were not used. Safety factors used in design of the primary elements in the NOFDS were insufficient (ASCE 2006a, 2006b). Quality assurance and control measures invoked during the life of the system failed to disclose critical flaws in the system (Vartabedian and Braun 2006). Inappropriate use was made of existing engineering technology available to design, construct, operate, and maintain a NOFDS that would have acceptable quality and reliability. Deficient risk management methods were used to allocate resources and impel action to properly manage risks (Moteff 2004). Risk management failed to employ continuing improvement, monitoring, assessment, and modifications in means and methods which were discovered to be ineffective (Senate Committee on Homeland Security and Governmental Affairs 2006).

12.8 References

- ASCE External Review Panel (2006a). External Review Panel Progress: Report Number 1, Letter to LTG Carl A. Strock, Chief, U.S. Army Corps of Engineers, Washington, DC.
- ASCE External Review Panel (2006b). External Review Panel Progress: Report Number 2, Letter to LTG Carl A. Strock, Chief, U.S. Army Corps of Engineers, Washington, DC, Mar. 23, 2006.
- Baecher, G. B., and Christian, J. T. (2003). *Reliability and Statistics in Geotechnical Engineering*, Wiley, Chichester, UK.

- Bea, R. G. (1990). *Reliability Based Design Criteria for Coastal and Ocean Structures*. National Committee on Coastal and Ocean Engineering, The Institution of Engineers, Australia, Barton ACT.
- Bea, R.G. (1996). "Reassessment and Requalification of Infrastructure: Application to Offshore Structures." *J. Infrastructure Systems*, (2)2, 22-32.
- Bea, R. G. (1998). "Reliability Characteristics of a Platform in the Mississippi River Delta." *J. Geotech. and Geoenv. Engrg.*, 124 (8), 729-738.
- Bea, R. G. (2000). "Performance Shaping Factors in Reliability Analysis of Design of Offshore Structures." *J. Offshore Mechanics and Arctic Engineering*, (122) 1.
- Bea, R. G. (2001). "Risk Assessment and Management of Offshore Structures." *Prog. Struct. Engrg. & Materials*, 3, John Wiley & Sons, Ltd., 180-187.
- Bea, R. G. (2006). "Reliability and Human Factors in Geotechnical Engineering." *J. Geotechnical and Geoenvironmental Engineering*, 132 (5), 631-643.
- Braun, S., and Vartabedian, R. (2006). "The Politics of Flood Control, Levees Weakened as New Orleans Board, Federal Engineers Feuded." *Los Angeles Times*, 12/25/06.
- Brouwer, G. (2003). "The Creeping Storm," *Civil Engineering*, ASCE, Reston, VA.
- Carter, N. T. (2003). *Army Corps of Engineers: Civil Works Reform Issues in the 107th Congress*, Congressional Research Service Report for Congress, Washington DC, April.
- Carter, N. T. (2005a). *Water Resources Development Act (WRDA): Army Corps of Engineers Authorization Issues in the 109th Congress*, Congressional Research Service Report for Congress, Washington DC, March.
- Carter, N. T. (2005b). *Corps of Engineers Reform in WRDA 2005*, Congressional Research Service Report for Congress, N. T. Carter, Washington DC, April.
- Carter, N. T. (2005c). *New Orleans Levees and Floodwalls: Hurricane Damage Protection*, Congressional Research Service Report for Congress, Washington DC, September.
- Carter, N. T. (2005d). *Flood Risk Management: Federal Role in Infrastructure*, Congressional Research Service Report for Congress, Washington DC, October.
- Carter, N. T., and Sheikh, P. A. (2003). *Army Corps of Engineers Civil Works Program: Issues for Congress*, Congressional Research Service Report for Congress, Washington DC, May.
- Carter, N. T., and Hughes, H. S. (2005). *Army Corps of Engineers Water Resources Activities: Authorization and Appropriations*, Congressional Research Service Report for Congress, Washington DC, February.
- Carter, N. T. et al. (2005). *Army Corps of Engineers Civil Works Program: Issues for the 109th Congress*, Congressional Research Service Report for Congress, Washington DC, June.
- Chatry, F. M. (1985). "NGS Benchmarks." *Letter to Commander, Lower Mississippi Valley Division, U. S. Army Corps of Engineers from New Orleans District Corps of Engineers, New Orleans*.

- Christian, J. T. (2004). "Geotechnical Engineering Reliability: How Well Do We Know What We Are Doing?" *J. Geotechnical and Geoenvironmental Engineering*, 130(10), 985-1003.
- Coastal Environments (1984). *The Mississippi River Gulf Outlet: A Study of Bank Stabilization*, Report to St. Bernard Parish Police Jury, National Oceanic and Atmospheric Administration and State of Louisiana Department of Natural Resources, Baton Rouge.
- Collins, S. M. (2005). "Hurricane Katrina: Who's In Charge of the New Orleans Levees?" *Opening Statement, Homeland Security and Governmental Affairs Committee*, Washington, DC, 12/15/2005.
- Collins, S., and Liberman, J. (2005). *Senate Homeland Security Committee Holds Hurricane Katrina Hearing to Examine Levees in New Orleans*, Press Release, Washington DC, Nov. 2.
- Duncan, J.M. (1970). *Strength and stress-strain behavior of Atchafalaya foundation soils*. Research Report TE70-1, Department of Civil Engineering, University of California, Berkeley.
- Duncan, M. J. (2000). "Factors of Safety and Reliability in Geotechnical Engineering." *J. Geotech. and Geoenv. Engrg*, 126 (4), 307-316.
- Edgers, L. et al (1973). *Undrained creep of Atchafalaya levee foundation clays*. Research Report R73-16, Soils Publication No. 319, Dept. of Civil engineering, Massachusetts Institute of Technology, Cambridge MA.
- Fischhoff, et al. (1981). *Acceptable Risk*, Cambridge University Press, UK.
- Foott, R., and Ladd, C. C. (1973). *The behavior of Atchafalaya test embankments during construction*. Research report R73-27, Dept. of Civil Engineering, Massachusetts Institute of Technology, Cambridge MA.
- Foott, R., and Ladd, C. C. (1977). "Behaviour of Atchafalaya levees during construction." *Geotechnique*, 27(2), 137-160.
- Government Accountability Office (1976). *Cost, Schedule, and Performance Problems of the Lake Pontchartrain and Vicinity*, Louisiana, Hurricane Protection Project, Report to the Congress, Washington DC, Aug. 31.
- Government Accountability Office (1983). *Water Project Construction Backlog - A Serious Problem With No Easy Solution*, Report to Committee on Public Works and Transportation, House of Representatives, GAO/RCED-83-49, Washington DC.
- Government Accountability Office (1997). *Results Act: Observations on the U.S. Army Corps of Engineers Civil Works Program's Draft Strategic Plan*, Report to Committee on Transportation and Infrastructure, House of Representatives, GAO/RCED-98-13R, Washington DC.
- Government Accountability Office (2003). *Corps of Engineers, Improved Analysis of Costs and Benefits Needed for Sacramento Flood Protection Project*, Report to Congressional Requesters, GAO-04-30, Washington DC.
- Government Accountability Office (2005). *Army Corps of Engineers, Improved Planning and Financial Management Should Replace Reliance on Reprogramming Actions to Manage*

- funds, Subcommittee on Energy and Water Development, Committee on Appropriations, House of Representatives, GAO-05-946, Washington DC.*
- Government Accountability Office (2005a). *Hurricane Protection: Statutory and Regulatory Framework for Levee Maintenance and Emergency Response for the Lake Pontchartrain Project*, Washington DC.
- Government Accountability Office (2005b). *Army Corps of Engineers, History of the Lake Pontchartrain and Vicinity Hurricane Protection Project*, GAO-06-244T, Washington, DC.
- Government Accountability Office (2005c). *Army Corps of Engineers, Improved Planning and Financial Management Should Replace Reliance on Reprogramming Actions to Manage Project Funds*, Report to Subcommittee on Energy and Water Development, Committee on Appropriations, House of Representatives, Washington, DC, Sept.
- Government Accountability Office (2006). *Preliminary Observations Regarding Preparedness and Response to Hurricanes Katrina and Rita*, GAO-06-365R, Washington, DC.
- Graham, H. E. and Nunn, D. E. (1959). *Meteorological Considerations Pertinent to Standard Project Hurricane, Atlantic and Gulf Coasts of the United States*, National Hurricane research Project, U. S. Weather Bureau, Report No. 33, Washington, DC.
- Harr, M. E. (1987). *Reliability-Based Design in Civil Engineering*, McGraw-Hill, New York.
- Heinzerling, L. and Ackerman, F. (2002). *Pricing the Priceless*, Georgetown Environmental Law and Policy Institute, Georgetown University Law Center, Washington DC.
- Hallowell, C. (2001). *Holding Back the Sea, The Struggle on the Gulf Coast to Save America*, Harper Perennial.
- Houck, O. (2006). "Can We Save New Orleans?" *Tulane Environmental Law Journal*, 19(1), <<http://www.law.tulane.edu/tuexp/journals>> (May 1, 2006).
- Interagency Performance Evaluation Task Force (2006a). *Performance Evaluation Status and Interim Results, Report 1 of a Series, Performance Evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System*, U. S. Army Corps of Engineers, Washington, DC.
- Interagency Performance Evaluation Task Force (2006b). *Performance Evaluation Status and Interim Results, Report 2 of a Series, Performance Evaluation of the New Orleans and Southeast Louisiana Hurricane Protection System*, U. S. Army Corps of Engineers, Washington, DC.
- Irons, L. (2005). "Hurricane Katrina as a Predictable Surprise." *Homeland Security Affairs*, 1(2), <<http://www.hsaj.org>> (April 1, 2006).
- Jordan, I. (2005). *Decisions Under Uncertainties, Probabilistic Analysis for Engineering Decisions*, Cambridge University Press.
- Kammen, D., and Hassenzahl, D. (1999). *Should We Risk It? Exploring environmental, Health, and Technological Problem Solving*, Princeton University Press.
- Kaufman, R. I., and Weaver, F. J. (1967). "Stability of Atchafalaya levees." *J. Soil Mechanics and Foundations Division*, 93(4), 157-176.

- Klotzbach, P.J. and Gray, W.M. (2006). *Extended Range Forecast of Atlantic Seasonal Hurricane Activity and U.S. Landfall Strike Probability for 2006*, Dept. of Atmospheric Science, Colorado State University.
- Knoll, F. (1986). "Checking Techniques." *Modeling Human Error in Structural Design and Construction*, A. S. Nowak, ed, 26-42.
- Kolb, C.R., and Van Lopik, J.R. (1958). *Geology of the Mississippi River Deltaic Plain, Southern Louisiana*. Technical Report No. 3-483, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.
- Krinitzsky, E. L., and Smith, F. L (1969). *Geology of backswamp deposits in the Atchafalaya basin, Louisiana*. Technical Report S-69-8, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.
- Kulhawy, F. (1996). "From Casagrande's Calculated Risk to Reliability-Based Design in Foundation Engineering." *Civil Engrg. Practice*, BSCE, 11(2), 43-56.
- Lacasse, S. (2004). "Risk Assessment for Geotechnical Solutions Offshore." *Proc. 23rd Int. Conf. Offshore Mechanics and Arctic Engrg.*, OMAE2004-51144, ASME.
- Ladd, C. C. et al. (1972). *Engineering properties of soft foundation clays at two south Louisiana levee sites*. Research Report R72-26, Dept. of Civil Engineering, Massachusetts Institute of Technology, Cambridge MA.
- Leonard, H.B. and Howitt, A.M. (2006). "Katrina as Prelude; Preparing for and Responding to Future Katrina-Class Disturbances in the United States." *Testimony before the U.S. Senate Homeland Security and Governmental Affairs Committee*, Washington, DC.
- Loosemore, M. (2000). *Crisis Management in Construction Projects*, ASCE Press, Reston, VA.
- Marshall, R. (2006). "Floodwall failure was foreseen, team says." *Times Picayune*, New Orleans, LA, 03/14/2006, <<http://www.nola.com>> (May 1, 2006).
- Members Scholars of the Center for Progressive Reform (2005). *An Unnatural disaster: The Aftermath of Hurricane Katrina*, Center for Progressive Reform, Report Publication No. 512, <<http://www.centerforprogressivereform.org>> (May 1, 2006).
- Molak, V. (Ed) (1997). *Fundamentals of Risk Analysis and Risk Management*, CRC Lewis Publishers, New York.
- Motef, J. (2004). *Risk Management and Critical Infrastructure Protection: Assessing, Integrating, and Managing Threats, Vulnerabilities and Consequences*, Congressional Research Service Report for Congress, Washington DC, 9/2/2004.
- National Academy of Engineering (2006). *Letter to Honorable John Paul Woodley, Assistant Secretary of the Army, Civil Works*, Committee on New Orleans Regional Hurricane Protection Projects, Washington DC, Feb. 21.
- National Research Council (1983). *Safety of Existing Dams, Evaluation and Improvement*, Committee on Safety of Existing Dams, Commission on Engineering and Technical Systems, National Academies Press, Washington, DC.
- National Weather Service (1979). *Meteorological Criteria for Standard Project Hurricane and Probable Maximum Hurricane Windfields, Gulf and East Coasts of the United States*, NOAA Tech. Report NWS 23, Charleston, SC.

- Netherlands Water Partnership (2005). *Dutch Expertise Water Management & Flood Control*. Delft, The Netherlands, <<http://www.nwp.nl>> (Dec. 1, 2005).
- Office of Management and Budget (2006). *Agency Scorecards, Corps of Engineers - Civil Works*, The Executive Office of the President, The White House, <<http://www.whitehouse.gov/omb>> (May 1, 2006).
- Oner, et al (1997). "Soil-Structure Interaction Effects in Floodwalls." *Electronic Journal of Geotechnical Engineering*, <<http://www.ejge.com/1997>> (Jan,1, 2006).
- Perdikis, H. S. (1967). "Hurricane Flood Protection in the United States." *J. Waterways and Harbors Div.*, ASCE, Reston, VA.
- Petroski, H. (1985). *To Engineer is Human: The Role of Failure in Successful Design*, St. Martins Press, New York, NY.
- Petroski, H. (1994). *Design Paradigms, Case Histories of Error and Judgment in Engineering*, Cambridge University Press, Cambridge, UK.
- Powers, K. (2005). *Aging Infrastructure: Dam Safety*, Congressional Research Service Report for Congress, Washington DC, September 2005.
- Rasmussen, J. (1997). "Risk Management in a Dynamic Society: A Modeling Problem." *Safety Science*, 27 (2), Elsevier, UK.
- Schwert, et al. (1979). *Meteorological Criteria for Standard Project Hurricane and Probable Maximum Hurricane Windfields, Gulf and East Coasts of the United States*, National Weather Service, Washington, DC.
- Seed, R. B., and Bea, R. G. (2006). *Initial Comments on Interim (70%) IPET Study Report*, National Science Foundation-Sponsored Independent Levee Investigation Team (ILIT), University of California, Berkeley, Mar. 12, 2006.
- Seed, R. B., et al. (2005). *Preliminary Report on the Performance of the New Orleans Levee Systems in Hurricane Katrina on August 29, 2005*, Report No. UCB/CITRIS-05/01, Berkeley, CA, <<http://risk.berkeley.edu>> (Nov. 17, 2005).
- Select Bipartisan Committee to Investigate the Preparation for and Response to Hurricane Katrina (2006). *A Failure of Initiative*. U.S. Government Printing Office, Washington, DC, <<http://www.gpoaccess.gov/congress/index.html>> (Mar. 15, 2006).
- Senate Committee on Homeland Security and Governmental Affairs (2006). *Hurricane Katrina: A Nation Still Unprepared*, Washington DC.
- Sevedung, I., and Rasmussen, J. (2002). "Graphic Representation of accident scenarios: mapping system structure and the causation of accidents." *Safety Science*, 40(1), Elsevier, UK.
- Shapira, Z. (1995). *Risk Taking, A Managerial Perspective*, Russell Sage Foundation, New York.
- Simpson, R. (Ed) (2003). *Hurricane! Coping with Disaster, Progress and Challenges Since Galveston, 1900*. American Geophysical Union, Washington DC.
- Sowers, G. F. (1993). "Human Factors in Civil and Geotechnical Engineering Failures." *J. Geotechnical Engineering*, 109(2).
- Spouge, J. (1999): *A Guide to Quantitative Risk Assessment for Offshore Installations*, Publication 99/100 CMPT 1999, ISBN I 870553 365, London, UK.

- Tekie, P. B., and Ellingwood, B. R. (2003). "Perspectives on probabilistic risk assessment of concrete gravity dams." *Applications of Statistics and Probability in Civil Engineering*, Der Kiureghian, Madanat and Pestana (Eds.), Millpress, Rotterdam.
- Townsend, F. F. (2006). *The Federal Response to Hurricane Katrina, Lessons Learned*, Report to The President of the United States, The White House, Washington, DC.
- URS Corporation (2005). *Interim Report: Hydrodynamic Modeling Effort for MRGO Study*, Report to State of Louisiana Department of Natural Resources, Baton Rouge, LA.
- U.S. Army Corps of Engineers, New Orleans District (1968). *Field tests of levee construction, test sections I, II, and III, EABPL, Atchafalaya Basin Floodway, Louisiana*. Interim Report, New Orleans, LA.
- U.S. Army Corps of Engineers (1974). *Final Environmental Statement, Lake Pontchartrain, Louisiana and Vicinity Hurricane Protection Project I-2*, New Orleans District, New Orleans, LA.
- U.S. Army Corps of Engineers (1988a). *Lake Pontchartrain, LA., and Vicinity Lake Pontchartrain High Level Plan, Design Memorandum No. 19 Orleans Avenue Outfall Canal*. Three Volumes, New Orleans District, New Orleans, LA.
- U.S. Army Corps of Engineers (1988b). *E-99 Sheet Pile Wall Field Load Test Report*. Technical Report No. 1, U.S. Army Engineer Division, Lower Mississippi Valley, Vicksburg, MS.
- U.S. Army Corps of Engineers (1989a). *Lake Pontchartrain, LA., and Vicinity Lake Pontchartrain High Level Plan, Design Memorandum No. 19A London Avenue Outfall Canal*. Two Volumes, New Orleans District, New Orleans, LA.
- U.S. Army Corps of Engineers (1989b). *Development of Finite-Element-Based Design Procedure for Sheet-Pile Wall*. U.S. Army Corps of Engineers Waterways Experiment Station, Tech. Report GL-89-14, Vicksburg, MS.
- U.S. Army Corps of Engineers (1990). *Lake Pontchartrain, LA., and Vicinity Lake Pontchartrain High Level Plan, Design Memorandum No. 20, General Design, Orleans Parish, Jefferson Parish, 17th. Outfall Canal (Metairie Relief)*. Two Volumes, New Orleans District, New Orleans, LA.
- U.S. Army Corps of Engineers (1994). *Design of Sheet Pile Walls*. Engineer Manual EM 1110-2-2504, <<http://www.usace.army.mil/inet/usace-docs>> (Apr. 1, 2006).
- U.S. Army Corps of Engineers (1996). *Risk-Based Analysis for Flood Damage Reduction Studies*. Engineer Manual EM 1110-2-1619, <<http://www.usace.army.mil/inet/usace-docs>> (Apr. 1, 2006).
- U.S. Army Corps of Engineers (1999). *Risk-Based analysis in Geotechnical Engineering for Support of Planning Studies*. Engineer Manual EM 1110-2-1902, <<http://www.usace.army.mil/inet/usace-docs>> (Apr. 1, 2006).
- U.S. Army Corps of Engineers (2000). *Design and Construction of Levees*. Engineer Manual EM 1110-2-1913, <<http://www.usace.army.mil/inet/usace-docs>> (Apr. 1, 2006).
- U.S. Army Corps of Engineers (2003a). *Slope Stability*. Engineer Manual EM 1110-2-1902, <<http://www.usace.army.mil/inet/usace-docs>> (Apr. 1, 2006).

- U.S. Army Corps of Engineers (2003b). *USACE 2012, Aligning the U.S. Army Corps of Engineers for Success in the 21st Century*, Washington, DC.
- Vartabedian, R. and Braun, S. (2006). "Unfinished 1965 Project Left Gaps." *Los Angeles Times*, 1/17/2006.
- Versteeg, M.F. (1988). "External Safety Policy in the Netherlands: an Approach to Risk Management." *J. of Hazardous Materials*, Elsevier 17, Amsterdam, The Netherlands.
- Vick, S. G. (2002). *Degrees of Belief, Subjective Probability and Engineering Judgment*, ASCE Press, Reston, VA.
- Weick, K. E., and Sutcliffe, K. M. (2001). *Managing the Unexpected*, Jossey-Bass, San Francisco, CA.
- Wenk, E., Jr. (1989). *Tradeoffs: Imperatives of Choice in a High Tech World*, The Johns Hopkins University Press, Baltimore, MD
- Wenk, E., Jr. (1995). *Making Waves: Engineering, Politics and the Social Management of Technology*, University of Illinois Press, Urbana, IL.
- Wenk, E., Jr. (1999). *The Double Helix: Technology and Democracy in the American Future*, Ablex Publishing Corp., Stamford CT.
- Whitman, R. V. (2000). "Organizing and Evaluating Uncertainty in Geotechnical Engineering." *J. Geotech. Engrg*, 126 (7), 583-592.
- Whitman, R. V. (1984). "Evaluating Calculated Risk in Geotechnical Engineering." *J. Geotech. Engrg*, 110 (2), 145-188.
- Wolff, T. (1999). *Evaluating the Reliability of Existing Levees*. ETL 1110-2-556, U.S. Army Corps of Engineers, <<http://www.usace.army.mil/inet/usace-docs>> (Mar. 1, 2006).
- Wolff, T. (2005). "Factors of Safety for Levee Design", Personal Communication.

CHAPTER THIRTEEN: ORGANIZED FOR SUCCESS

The excuse we have heard from some government officials throughout this investigation, that Katrina was an unforeseeable ultra-catastrophe, has not only been demonstrated to have been mistaken, but also misses the point that we need to be ready for the worst that nature or evil men can throw at us. Powerful though it was, the most extraordinary thing about Katrina was our lack of preparedness for a disaster so long predicted.

This is not the first time the devastation of a natural disaster brought about demands for a better, more coordinated government response. In fact, this process truly began after a series of natural disasters in the 1960s and into the 1970s. One of those disasters was Hurricane Betsy, which hit New Orleans in 1965. The similarities with Katrina are striking: levees overtopped and breached, severe flooding, communities destroyed, thousands rescued from rooftops by helicopters, thousands more by boat, and too many lives lost.

Katrina revealed that this kaleidoscope of reorganizations has not improved our disaster management capability during these critical years. Our purpose and our obligation now is to move forward to create a structure that brings immediate improvement and guarantees continual progress. This will not be done by simply renaming agencies or drawing new organizational charts. We are not here to rearrange the deck chairs on a ship that, while perhaps not sinking, certainly is adrift.

This new structure must be based on a clear understanding of the roles and capabilities of all management agencies. It must establish a strong chain of command that encourages, empowers, and trusts frontline decision-making. It must replace ponderous, rigid bureaucracy with discipline, agility, cooperation, and collaboration. It must build a stronger partnership among all levels of government with the responsibilities of each partner clearly defined, and it must hold them accountable when those responsibilities are not met.

Senator Susan Collins

Opening Statement

Committee on Homeland Security and Government Affairs

Hurricane Katrina: Recommendations for Reform

Washington DC, March 8, 2006

We are Doomed to an Unacceptable Future - Unless ...

What do the following accidents have in common? Torrey Canyon tanker (1967) and the Exxon Valdez tanker (1989); the U.S.S. Greenville (2002) and the U.S.S. San Francisco (2005); the Challenger Space Shuttle (1986) and the Columbia Space Shuttle (2003); the Piper Alpha Platform (1988) and the Petrobras P36 Platform (2001), Herald of Free Enterprise (1987) and the Estonia ferry (1994), and failure of the NOFDS in the wake of hurricane Betsy (1965) and hurricane Katrina (2005).

In each case someone, somewhere, understood that organizational and system processes were as much the cause of the accident as were engineering design, construction and maintenance errors (Appendix F). In each case this knowledge failed to prevent a second disaster from happening in the same industry. This record suggests that we are doomed to a future in which increasingly complex organizations and systems of organizations fail causing unnecessary death and injury, large scale economic disruption, political haggling, and years of rebuilding.

We are doomed to this future despite growing evidence that preventing disasters is always cheaper than recovery. We are doomed to this future despite the fact that we know that technological failures virtually always occur within the context of management failures, and there is a growing body of literature that describes management implementations designed to reduce large scale failure (e.g. Roberts and Bea 2001a; 2001b; Dekker 2002; Weick and Sutcliffe 2001).

As an example of what doesn't happen, the National Incident Management System (NIMS) Integration Center issued this alert (Department of Homeland Security 2006):

All federal, state, local, tribal, private sector and non-governmental personnel with a direct role in emergency management and response must be NIMS and ICS trained. This includes all emergency services related disciplines such as EMS, hospitals, public health, fire service, law enforcement, public works/utilities, skilled support personnel, and other emergency management response, support and volunteer personnel....

In mid March, 2005, Donald Hiett, Jr, Principal, Organizational Strategic Solutions Group, was asked by Louisiana State University (LSU) to develop a NIMS training program directed to the senior executive leadership in New Orleans to take place before June, 2006. On March 28, 2005 he was informed there was no interest by these officials in taking this training program (Hiett, personal communication).

We are doomed – unless. This chapter deals with “unless.” It first discusses the assessment of safety, and the usual engineering responses to risk. It then asks that the reader adopt a new perspective regarding the USACE and the contextual issues it needs to consider. It then discusses preventing the “next Katrina”, and offers recommendations.

13.1 How Safe is Safe Enough?

The hurricane Katrina catastrophe exposed a technological failure of inadequate defenses against a predictable, risky and potentially lethal event. Recent studies have tended to focus primarily on death and destruction from flood waters released by collapse of the NOFDS. Studies of cause acknowledge the extreme forces of nature, but also cite human and organizational errors (HOE) that now occur more conspicuously because the engineering parameters are fairly well understood. HOE failures far exceed mechanical sources in the overall Katrina catastrophe.

Because protection against human weaknesses is more art than science, the study of the causes and remediation of HOE require a context for risk analysis. Non-specialists with policy and management responsibilities should be helped by a perspective that points to the systems-based and interdisciplinary requirements for the NOFDS. Such a perspective can help us answer the enigmatic question, “How Safe is Safe Enough?” In other words, what level of risk is acceptable when making decisions about public safety and security?

Risk is usually defined as a condition in which either an action or its absence poses threats of socially adverse and sometimes extreme consequences. Risk happens from acts of nature, from weaknesses of human nature, and from side effects of technology, all situations that mix complex technical parameters with the variables of social behavior. Although each risk event is unique, all display commonalities that permit systemic analysis and management. These recurring properties lead to certain principles.

To begin, the acceptability of risk cannot be extracted from science or mathematics; it is a social judgment. The spectrum of risk thus embraces both the physical world defined by natural laws, and the human world loaded with beliefs instead of facts, and with values, ambiguities and uncertainties. Among other features, the physical world may be thought of as a mechanism whose behavior follows principles of cause-and-effect. The human world performs more like an organism whose components are not fixed but may grow, and which may be altered by the thrust of events and their interplay with other elements.

Following a notion that what you can’t model you can’t manage, a systems model is needed to represent the processes by which both physical and societal factors are defined, interconnected and interact. Such technology-based human support systems are labeled by their intended social functions: food production, shelter, military, homeland security, etc. In our modern era, these and other functions are enormously strengthened by applications of scientific knowledge, applied through engineering.

It helps to think of technology as more than the hardware of planes, trains and computers. Rather, it is a social system comprising many organizations, synchronized by a web of communications for a common purpose. It is energized by forces of free market demand, of popular demand for security and quality of life, and by forces of scientific discovery and innovation. It is best understood as a *Technological Delivery System* (TDS) that applies scientific knowledge to achieve society’s needs and wants.

Technology then acts like an amplifier of human performance. Like the water wheel, the steam engine and the bomb, it amplifies human muscle. With the computer it amplifies the human mind and memory. It also amplifies social activity, mobility, quality and length of life.

A paradox arises when technologies introduced for specific benefits also spawn side effects. These can induce complexity, conflict and even chaos. Most of these are unwanted by

some sector of stakeholders, now or in the future. This paradox is dramatized when technologies are introduced to defend against violence of nature, or against human and organizational error, but themselves spring unintended and possibly dangerous consequences.

The investigation of risk and of measures to contain it within safe limits requires both hindsight and foresight. The past can illuminate failures, their causes and their control as lessons for engaging new issues and threats. The future commands the exercise of foresight, an imaginative preparation of scenarios stirred by such questions as, “what might happen, if?” or “what might happen, unless?” Those inquiries should then examine the timing of impacts (immediate or hibernating), the identities of players on the risk horizon who may trigger risk, and those parties responsible for risk abatement and those who may be adversely affected now or in the future.

Modeling then becomes essential to represent a full cast of stakeholders and their inter-relationships, including both the private and the public sectors. The concept of a technology delivery system (TDS) is simply an attempt to model how the real world works.

The responsibility to manage risk stems from the American Constitution, from custom, and from a growing body of public law. Federal, state, and local governments are heavily involved in all of the technologies previously discussed and many more. With waterways, for example, the Army Corps of Engineers (USACE) has a predominant statutory responsibility. That accords with the historic federal stewardship of national infrastructure, from roads, shipping channels, harbors and canals to airplane routes and the Internet.

That achievement carries significant but subtle implications. For one thing, safety costs money. The federal budget is constantly challenged to meet a rainbow of different demands, the total of which always exceeds Congressional appropriations. The mismatch must then be reconciled through tradeoffs at the highest policy levels stretching all the way to the President of the United States and the Congress.

Often, a focus on power of the Federal Government misses a major premise of democratic governance. As the Declaration of Independence states, *those who govern should do so only with the consent of the governed*; we would say the informed consent. This notion is reflected in such regulatory legislation as the National Environmental Policy Act (NEPA), Section 102(2) c. It requires estimates of harm that could result from technological initiatives, along with alternatives to accomplish the same goals but with less harm. After preparation, these environmental impact statements (EIS) are made available for public comment and possible amendment. The point is that this process makes every citizen a part of government process to negotiate the question of how safe is safe enough and thus provide citizens the levels of safety and security that they desire.

Implied is a prospective national policy that those put in harm’s way have a voice in what otherwise could be involuntary exposure to risk. This principle leaves implementation of the concept to the responsible federal agencies, subject to Constitutional safeguards. Despite a tendency to flare the sensational, the media can enrich understanding with a backstory because disasters so agitate a functioning system as to reveal the full cast of stakeholders, their roles in increasing or decreasing risk and their degree of injury.

In this modern era, society demands better protection against threats to life, peace, justice, health, liberty, lifestyle, private property and to the natural environment. These challenges are not new, but two things have changed—the increased potency of technology and

increased media coverage. Technological factors are more robust in speed of delivery and in potential harm. Media covers events live, 24/7, and worldwide. Events anywhere have repercussions everywhere. The better informed public tends increasingly to be risk averse. Apprehension and fear peak after a calamity with demands for better protection through better governance. Higher expectations are legitimate because so many threats just itemized are due to human and organizational errors either in catering technologies to meet market demand or in guarding against hazards. This current study shows that the Katrina event fits that pattern. Government at all levels failed to provide security to citizens before and during the catastrophic flooding. Victims are justified in asking how this pathology of a mundane levee technology developed; How can that knowledge be applied to prevent a reoccurrence?

13.1.1 The Engineering Response to “How Safe is Safe Enough?”

The engineering profession has long practiced social responsibility by a technique of over-design, to compensate for uncertainties in loading, in materials, in quality of construction and maintenance, etc. This may be accomplished by adopting some multiple of loading as a margin of safety ranging from 1.4 to 5.0 and even greater. How these margins are set, and by whose authority, is of critical importance; especially where tradeoffs with cost or other compelling factors such as deadlines may compromise the intended reduction of risk.

This method of safety assurance is more applicable to design of mechanisms not subject to human and organizational errors. The term “errors,” incidentally, is shorthand for a broad spectrum of individual and societal weaknesses that include ignorance, blunder, folly, mischief, pride, lack of competence, greed and hubris. Protecting structures against violence of nature such as with earthquakes, volcanic eruptions, tsunamis, floods, landslides, hurricanes, pestilence, droughts and disease may utilize the concept of over-design, based on meteorological, hydrological, seismic and geophysical data of past extreme events.

Learning from documented failures is a powerful method for reducing risks of repeated losses. Another method is to learn from close shaves. Many dangerous events fortunately culminate in only an incident rather than an accident, but the repetition of similar incidents can serve as early warnings of danger. Indeed, the logging and analysis of such events on the nation’s airways partially accounts for commercial aviation’s impressive safety record. A system for reporting close encounters of aircraft was installed decades ago. Anticipating the possibility that perpetrators of high risk events might be reluctant to blow the whistle on themselves, many years ago the Federal Aviation Administration arranged for NASA to collect incident data and to sanitize it to protect the privacy of the incident reporter. NASA also screened reports to identify patterns as early warning of dangerous conditions. Similar systems are in place for reporting nuclear power plant incidents.

With the growing recognition of human factors in accidents or in failures to limit damage, a class of situations entailing uncommonly high risks but conspicuously good safety records was examined. In the Navy, for example, high risks are a part of daily operations of submarines and aircraft carriers. Yet accident rates are paradoxically low. Careful analysis of these situations showed that certain qualities of leadership and organizational culture foster integrity, a sense of responsibility among all participants, a tolerance by authority figures for dissent, and consensus on common goals of safe performance. High safety performance is associated with an institutional culture that is bred from the top of the management pyramid. The most critical element of that culture is mutual trust among all parties (e.g., Roberts 1990).

Long experience with military and paramilitary organizations such as first responders proves the value of rehearsals to reduce risks and control damage. Of special virtue is proof of satisfactory communications. Evaluation of dry runs has repeatedly turned up serious problems in communication. So has post-accident analysis of real events when delays or blunders in communication of warnings and rescue operations cost lives.

13.1.2 Insights from Addressing These Issues

To sum up, the context for analyzing the levee failures from Hurricane Katrina illustrates several realities. The most compelling imperative of life is survival. Yet the experience of living teaches that there is no such thing as zero risk. Some exposures must be tolerated as “normal,” whether in rush hour traffic or when coping with nature, with human nature or with unintended consequences of technology. The preceding situation analysis opens a window on a number of issues treated in more detail in subsequent sections and Appendix H, including the following:

- The design of precautionary measures requires inspired foresight, to imagine or foresee alternative futures.
- Tradeoffs are inevitable between short- and long-range events and consequences, between safety and cost, between special interests and social interests, between who wins and who loses, and who decides.
- All human support systems entail technology, and all technologies project unintended consequences.
- Society embraces a spectrum of values that often conflict, as with the goals of efficiency in the private sector and of sustainability and social justice in the public
- Key decisions regarding citizen safety and security are made by government through public policies to manage risk. These policies dominate the legislative agenda.
- This mandate imposes a heavy burden on the President and on Congress, both bodies requiring access to authentic and immediate information.
- Making decisions and assuring implementation draws on political capital in the structure of authority by the exercise of political power and political will.
- In our democracy, this authority should flow from citizens following the principle that those who govern do so at the informed consent of the governed.
- The quality of risk management can best be judged by the effects on future generations.
- The geography of risk crosses boundaries between federal, state and local entities, and also between the United States and other nations.
- Different cultures have different risk tolerances, including attitudes distinguishing voluntary from involuntary risk.
- Analysis of risk and its control extracts lessons from past failures, although the most catastrophic events are so rare as to often frustrate projections.

This portfolio of issues illustrates the anatomy of risk and the complexity of its management. They sound a wake-up call for deeper understanding by those responsible for risk

management, and by those attentive citizens who are exposed and are entitled to a voice in the decision process.

13.2 Maximizing How Safe is Safe in the U.S. Army Corps of Engineers (Context)

We know a finite number of precursors lead to major disasters. But in order to understand what they are we must place a focal first responder organization into its context. For example, the Corps of Engineers (USACE) is nested within a large number of organizations that should be interdependent with one another. The social science literature addresses this problem by using such concepts as *interstices* (Grabowski and Roberts 1999), “interdependencies” (Heath and Staudenmayer 2000) or the “space between” (Bradbury and Lichtenstein 2000; Buber 1970). Failure to consider the processes that operate both within any one unit and across multiple units is failure to be ready for the next large scale catastrophe. This discussion focuses on context and asks the reader to take a new perspective of the Corps of Engineers.

Hurricane Katrina provided an interesting, if devastated setting for understanding what not to do in a quickly changing potential disaster. The organizational liquefaction that occurred after the Hurricane (the heart of the disaster as opposed to the storm), laid bare the skeletons of the organizations that should have had flesh and muscle to respond. It laid bare for the public to see, not only skeletons, but complete organizational disregard for the interdependences so necessary to a coordinated response. As Houck (2006) observes:

So What Do We Do? Here is what we know. It is not just the tire, it's the car. And it's not just the car, it's the driver. Nothing in the system has made a numero uno priority either of protecting New Orleans from hurricanes or to restoring or even hanging onto - the Louisiana coast. We have a flood control program, a navigation program, a permitting program, a coastal management program, a flood insurance program, a coastal restoration program - just for openers - and they do not talk to each other. They are riddled with conflicts, basically headless, basically goal-less, weakened by compromises and refuse outright to deal with first causes and first needs.

The key phrases here are “and they do not talk to each other” and “They are riddled with conflicts, headless, basically goal-less...and refuse outright to deal with first causes....”

In reaction to the organizational liquefaction that developed during hurricane Katrina the Senate Committee on Homeland Security and Governmental Affairs recommended (2006):

The Corps and local levee sponsors should immediately clarify and memorialize responsibilities and procedures for the turn-over of projects to local sponsors, and for operations and maintenance, including, but not limited to procedures for the repair or correction of levee conditions that reduce the level of protection below the original design level (due to subsidence or other factors) and also emergency response. It must always be clear - to all parties involved - which entity is ultimately in charge of each state of each project. The Corps should also provide real-time information to the public on the level of protection afforded by the levee system. A mechanism should be included for the public to report potential problems and provide general feedback to the Corps.

13.2.1 The Office of the President, the Congress, and the Corps

Other things happen at interstices. Figure 13.1 shows the Presidential and Congressional budget requests and Congressional recommendations for Corps of Engineer funding for 1975 through 2005 for the Lake Pontchartrain and vicinity hurricane flood defense projects.

Several hypotheses can be gleaned from this information. First, it appears that while the president was trying to reduce Corps funding Congress was trying to protect Corps funding. With the Lake Pontchartrain projects only about sixty percent complete as of 2005 (40 years after authorization) it may be that Congress, in its wisdom, decided to fund only what it thought needed to be completed. The graph shows other interesting issues about interdependencies. The Corps of Engineers is interdependent with both the Office of the President of the United States and Congress. Congressional members bring pressure to bear on the Corps for new large projects. Faced with these pressures the Corps, then, defers maintenance. For over a decade Congress has funded the Corps at higher levels than recommended by the President. The Corps, then, has to devote time to currying favor with Congress. Currying favor with Congress is not supposed to be a main task of the Corps.

Yet another interesting hypothesis can be derived from these data. When multi-year projects are funded annually an interesting dilemma is created for the funded organizations. The funding oscillation level is at one level, but organizations struggling under that oscillation oscillate at a higher frequency. It is hypothesized that this is because the funded organization operates under a considerable amount of ambiguity and uncertainty. This suggests that the unpredictability of the Congressional process creates unintended and negative consequences for its funded agencies. The processes and responses to them are both schizophrenic.

This is almost surely the same as the case for NASA. The Columbia Accident Investigation Board (CAIB) report said (Columbia Accident Investigation Board, 2003):

The White House and Congress must recognize the role of their decisions in this accident and take responsibility for safety in the future.... Leaders create culture. It is their responsibility to change it.... The past decisions of national leaders – the White House, Congress, and NASA Headquarters – set the Columbia accident in motion by creating resource and schedule strains that compromised the principles of a high risk technology organization.

Diane Vaughan reports that both economic strain and schedule pressure still exist at NASA. She notes that it is unclear how the conflict between NASA's goals and the constraints upon achieving them will be resolved but that one lesson from Challenger and Columbia is that system effects tend to reproduce (Vaughan 2005). This also happens to military installations every time a Base Reallocation and Closing (BRAC) list is formed. From the day of its publication until the day of decisions, the installations on this list spend considerable time trying to get off the list, distracting them from their principle tasks.

In the Katrina case, will Congress and the Office of the President take a sweeping look at their own behaviors in concert with those of the Corps of Engineers? They probably will not because there is not yet a stated strong incentive for them to do so. One incentive might be that the cost of cleanup is always more than the cost of prevention. Money is not limitless. But since we've observed many costly past disasters that were not prevented, and many instances in which they could have been mitigated or prevented, the reality is they probably will do nothing. Thus, the challenge is to find incentives that will encourage both disaster prevention and emergency

response organizations, from the President on down, to examine their own organizational skeletons, muscle, and flesh, as well as to look at the “spaces between.”

13.2.2 Additional External Interstices for the Corps

Three additional sorts of interfacing between the USACE and its constituents need to be thought about. The first are the interfaces mandated by Emergency Support Function # 3 of the National Response Plan - NRP (Department of Homeland Security, 2004a).

ESF #3 is structured to provide public works and engineering-related support for the changing requirement of domestic incident management to include preparedness, prevention, response, recovery and mitigation actions. Activities within the scope of this function include conducting pre- and post-incident assessments of the public works and infrastructure; executing emergency contract support for life-saving and life-sustaining services; providing technical assistance to include engineering expertise, construction management, and contracting and real estate services; providing emergency repair of damaged infrastructure and critical facilities; and implementing and managing the DHS/Emergency Preparedness and Response/Federal Emergency Management Agency (DHS/EPR/FEMA) Public Assistance Program and other recovery programs.

To accomplish these goals, USACE can draw on the resources 15 federal government agencies. In addition, state, local and tribal governments are “fully and consistently integrated into EFS #3 activities.” (Department of Homeland Security, 2004a). All of this occurs, of course, when an incident or potential incident overwhelms state, local, and tribal capabilities.

The NRP concept of operations states that the DOD/USACE is the primary agency for providing ESF #3 technical assistance. It further states that close coordination is to be maintained with federal, state, local, and tribal officials to determine potential need for support. In addition it spells out the organizational structures for providing support, naming the Interagency Incident Management Group (IIMG) as the resource for providing on-call subject-matter experts to support IIMG activities.

Regional and field level mechanisms of support are clearly defined. ESF #3 activities are also spelled out and include such processes as:

coordination and support of infrastructure risk and vulnerability assessments, participation in pre-incident activities, such as pre-positioning assessment teams,... participation in post-incident assessments of public works and infrastructures to help determine critical needs and potential workloads, implementation of structural; and non structural mitigation measures, including deploying protective measures to minimize adverse effects or fully protect resources, prior to an incident.

In the wake of hurricane Katrina, neither the USACE nor any other agency was fully successful in rolling out the NRP. If the integration required by this plan is too difficult for agencies to implement, then it is the duty of the agencies and their oversight agencies (e.g.: DOD, DHS, HHS, etc.) to indicate this and to develop strategies to revise the NRP to create a workable plan and document. Lee Clarke (1999) discusses at length “fantasy plans” and that looks to be exactly what we have here. Thus, a last word on integration across agencies (Lakoff 2006):

From the vantage of preparedness, the failed response to Hurricane Katrina did not undercut the utility of “all-hazards” planning. Rather, it pointed to problems of implementation and coordination. This suggests that in the aftermath of the event, we are likely see the redirection and intensification of already-developed preparedness techniques rather than a broad rethinking of the security question.

Given our experiences with accident response, without substantial leadership and reorganization it is this team’s conclusion that neither comprehensive technical nor social reforms will likely soon be developed to address future natural or man made catastrophes.

The second set of interfaces that need to be thought about are those created by the Corps’ needs to “outsource” (the hiring of outside, private firms and/or individuals to perform work, including engineering design and construction.) The requirement for the Corps to do this has been imposed by the federal government; specifically through the White House Office of Management and Budget and through Congressional actions. Input from current Corps of Engineers personnel in multiple settings and private briefings with our team, clearly has indicated that through outsourcing and diversion of efforts the USACE has lost “engineering” (Figure 13.2). Core engineering (practicing, research, development) competencies have been sacrificed to pressures to outsource, to improve project management, and to develop environmental restoration and mitigation capabilities, all within a finite overall budget and resources.

Partnering has a number of advantages and disadvantages. Some operational benefits accrue from partnering. One can learn new things from partners, perhaps through access to best-of-class processes. Perhaps partnering competitors can learn technology secrets from one another. Where industry benchmarks aren’t well known, partnering with a competitor can offer insights on a company’s productivity, quality, and efficiency.

But there are also obvious disadvantages. Lack of control is a critical disadvantage. The demise of ValuJet, for example, happened because the company outsourced cargo handling to a company it had no control over in terms of quality standards. In another form of outsourcing, competitors learn from each others’ operations, which may be detrimental to one or more partners. Or a “coopetition” (combination of cooperation and competition) may self-destruct before the renewal option dates arrive. A new company board for one of the partners may not approve of the other partner. The strategic aims of partners may change mid-stream, causing failure. These are just some of the reasons for outsourcing failures. (Roberts and Wong, 2006). The Corps needs to examine its partner relationships, asking itself if it has lost too much.

One of the Corps sister agencies in time of chaos, FEMA, has also created problems through outsourcing its disaster response efforts (Perrow 2005):

For example, when the Nisqually earthquake struck the Puget Sound area in 2001, homes that had been retrofitted for earthquakes and schools with FEMA funds were protected from high-impact structural hazards. The day of that quake was also the day that the new president, G. W. Bush, chose to announce that Project Impact would be discontinued (Holdeman 2005). Funds for mitigation were cut in half, and those for Louisiana were rejected. Disaster management was being privatized, with the person who was to be promoted to head the agency, Michael Brown, saying at a conference in 2001, “The general idea—that the business of government is not to provide services, but to make sure that they are provided—seems self-evident to me” (Elliston 2004). The administration tried to

cut federal contribution for large-scale natural disaster expenditures from 75 percent to 50 percent.

13.2.3 The Corps Internal Interstices

Two other organizational processes also result in lost institutional memory and loss of control. They are downsizing and retirements. Table 13.1 shows that in recent years the USACE has lost employees. Figure 13.3 shows that the Corps is also losing employees through retirements. Recently, we were told by a high ranking official of the Corps that during the next 5 years, the Corps expects to lose approximately 40 to 50% of its civilian workforce through retirements.

In 2002, between 35 and 40 percent of architecture and engineering work was outsourced to private firms, while all construction projects were outsourced (U.S. Army Corps of Engineers 2002). The simultaneous operation of the three processes (outsourcing, downsizing and retirement) have been and will be disasters for the Corps. Retirements, downsizing, and outsourcing are interdependent in terms of the problems they cause for organizations. Again, the causes are probably buried in not only the Corps activities, but in the Corps' relationships with its external constituencies.

New approaches to looking at organizational failure examine the degree to which organizations are internally stove-piped. Figure 13.4 shows that the Corps organizational structure might lend itself to this. It appears regions and districts act pretty autonomously.

In addition Houck (2006) observes:

...restoring coastal Louisiana is a national issue and will require remedies beyond this state. We lie at the receiving end of a large watershed, and some of what we need has been turned off and other stuff that is hurting us has been turned on. The Corps districts need to talk to each other. The EPA has to step up to the plate, upstream states have to change some habits too. If the nation's taxpayers are going to be asked to spend more money than America spent on the Marshall Plan to fix all of post-war Europe, then they have a right to expect a national effort.

McCurdy (1993) discusses how stove-piping existed when NASA was created. Today the adverse results of NASA's stove-piping are excessive unit independence, specialization and neglect of mutual coordination in a situation that should be characterized by just the opposite (Roberts et al., 2005).

All in all, the Corps ability to do its job has been organizationally handicapped. It has lost engineering and research and development muscle and flesh, it has lost its ability to maintain old projects, it fails to be appropriately interdependent with various constituencies, and it fails to act effectively on issues of internal interdependence. **And, it cannot get well on its own.**

13.4 Preventing the Next Katrina

In virtually all human affairs, risk is normal. The consequences of neglect may be grave, if not now, in the future. As we indicated in the beginning of this chapter we are skeptical that

those with power and resources to prevent the next Katrina will take the steps necessary to do so and we provided evidence for this assertion

From our larger discussion about defining safety and including all stake holders in definition and response, three recommendations emerged:

- Responsibilities for vigilance and decision making at the tip of the authority structure should be clarified and strengthened to enhance management of all modes of risk.
- Additional technical Congressional staff should be appointed to assure adequate revenues to manage risk and to monitor performance of the Executive Branch in its duties of care.
- New processes should be authorized at a local level to foster informed consent and dissent, and to function as early warnings in disaster-prone areas, and to reflect that citizens at risk are entitled to information regarding their exposure and opportunities to participate in governance.

One central purpose should animate all the entities involved, separately and in tandem. They should address the question, “How Safe is Safe Enough?” That investigation demands foresight in the spirit of the injunction, “Without vision, the people perish.”

In addition to this larger purview, specific attention needs to be given to the Corps and the organizations with which it is interdependent. We know a great deal about how to fix problems of this nature, and there are growing bodies of engineering, legal, public policy, organizational, and other literatures that address such issues. There is also a growing body of experts from different areas who know how to talk about such issues. The problem is that stakeholders have huge incentives not to pay any attention to this. They are no more likely to fix this problem than they were likely to prevent the Challenger problem from becoming the Columbia problem or the Betsy problem from becoming the Katrina problem.

Fixing the problem will require a set of processes that affected stakeholders do not want to engage in:

- They must come together to decide exactly what they want (clear and consistent goals) in a politically complex and charged world.
- They must be willing to spend many years addressing such problems in a world in which incentives result in attention spans that more typically run the gamut of minutes to weeks.
- Agencies must work together and trust one another.
- They must recognize the interdisciplinary nature of their problems.
- They must be willing to spend money and make recipients of that money accountable for their spending.
- They must develop oversight programs and agencies with real teeth.

13.5 Re-Engineering the USACE

Fixing the USACE's technical problems will have only limited impact unless we also fix the organizational problems. The USACE must strive to become a High Reliability Organization - HRO (emulating the Rickover Navy; see Appendix G). Four recommendations that would go a long way toward repairing the Corp's ability to design and build effective flood control projects:

- Rebuild the USACE's engineering and R&D capability,
- Restructure the federal/state relationship in flood control,
- Develop a National Flood Defense Authority,
- Create effective disaster planning.

Three years before Katrina, the National Research Council concluded that the "Corps' more complex planning studies should be subjected to independent review by objective, expert panels." (National Research Council 2002). This is an obvious point – which makes it all the more urgent to implement. Although the need for independent project review has been apparent for years, none of the past proposals have yet been implemented.

13.5.1 Rebuilding USACE Technical/Engineering Capacity

The USACE's engineering and R&D capabilities were degraded over the past twenty years as a result of streamlining and budget cuts (downsizing and outsourcing). As a nation, we cannot afford the loss of this expertise. Although outsourcing can be efficient in some instances, it cannot be allowed to deplete USACE's own core expertise. As the National Research Council concluded, "Shifting analytical tasks to the private sector, however, has its limits, as core, "in-house" competence is necessary for the Corps to commission, manage, and comprehend the advice of external experts." (National Research Council, 2004)

The Army Corps of Engineers must be, first and foremost, the nation's premiere expert in flood control engineering. Through no fault of its own, the Corps has been stripped of much of what it needs to perform this role. Congress must adopt a plan and allocate the necessary funds to "put the 'engineers' back into the Corps of Engineers." It must remake the Corps into the organization that new, "wet behind the ears" civil engineers will want to join to sink their teeth into their new profession. It must retain and perform sufficient challenging engineering work as to encourage these engineers to develop their careers within the USACE. It must define and perform sufficient R&D work to help support the activities of these engineers. And it must pay them adequate salaries as to be suitably competitive with private industry.

The Working Group for Post-Hurricane Planning for the Louisiana Coast has advanced some complimentary recommendations for Corps staffing in their report *A New Framework for Planning the Future of Coastal Louisiana after the Hurricanes of 2005* (2006):

An essential element in enhancing the credibility and soundness of planning and implementation is an agency's internal staff capabilities. The Corps of Engineers is facing a significant loss of staff numbers and capability through retirement, just at the time that the demands for its skills are increasing. Indeed, the integrated planning process will demand a wider array of skills from the engineering, hydrologic, geological, biological and social sciences than is currently available in the agency or in federal or state agencies generally. Also, the effectiveness of

the long-term program requires the institutional memory that develops within a permanent and professional staff..

13.5.2 Restructuring the Federal/state Relationship in Flood Defense

The USACE's relationship with local flood control entities in Louisiana is dysfunctional. Some of the issues relate to the fragmentation of the local entities, which the state has begun to address. However, a number of the issues are broader.

Often, water planning activities involve not only multiple federal agencies, but also state and local governments. In the blunt words of one observer, "The first consequence is that flood defense has no head Whatever the merits of this diffusion of authority, it does not produce coherent flood control." (Houck, 2006). One useful model may be what has been called "modularity" -- a concept which involves provisional and functional rearrangement of units in terms of alternative configurations of tools, structures and relationships. (Freeman and Farber, 2005).

13.5.3 Developing a National Flood Defense Authority

A National Flood Defense Authority (NFDA) might be instituted and charged with oversight over the construction and maintenance of flood control systems. Each state would have an equivalent organization that could foster cooperation and developments between and within the states. The Corps of Engineers, state flood control authorities, and technical advisory boards would work with the NFDA to foster application of the best available technology and help coordinate development and maintenance efforts and planning. Federal and state governments would provide reliable and sustainable funding for the life-cycle (design, construction, operation, maintenance) of specific flood defense systems. To facilitate coherent funding, Congressional authorization and financing would be separated from the traditional Water Resources Development Act process.

The Corps of Engineers, in cooperation with other qualified agencies and industrial partners, would have the responsibility to design and construct, and if directed and authorized, operate and maintain flood defense systems. The NFDA would be based on a continuous and integrated process of flood risk assessment and management for specified flood defense systems, with each of these systems being integrated with other allied flood defense systems. Flood risk assessment and management processes would include proactive, reactive, and interactive (adaptive) approaches based on the best available proven technology. Flood defense system planning and development would engage public and industrial stakeholders and responsible federal and state agencies in a cooperative and vigilant Technology Delivery System.

The Interagency Floodplain Management Review Committee in 1994 advanced similar concepts as a result of their in-depth evaluation of the performance of existing floodplain management programs following the disastrous 1993 Midwest flooding. The Working Group for Post-Hurricane Planning for the Louisiana Coast has advanced similar recommendations for organization and funding in their report *A New Framework for Planning the Future of Coastal Louisiana after the Hurricanes of 2005*. This group observed (2006):

Organizational and funding barriers that have inhibited the adoption of an integrated planning and adaptive decision making process persist. Both new organization and funding reforms are needed to support coastal planning and project implementation by the Corps and the State.

This group proposed a model that involves proposals for Federal Intragovernmental Coordination, development of working processes with the new Louisiana Coastal Protection and Restoration Authority, the development of a Coastal Assessment Group and Coastal Engineering and Science Program. This model includes recommendations for programmatic authorization and funding including formation of a new Louisiana Coastal Investment Corporation and major revisions in the Water Resources Development Act appropriations process.

13.5.4 Creating Effective Disaster Planning

Research on organizational learning finds that practices and routines in organizations develop incrementally through feedback from the organization's environment. Organizations generally tend to be inert, adapting less than perfectly to and falling in and out of alignment with their environments (Nelson and Winter, 1982).

This stagnation is especially dangerous for organizations that deal with major emergencies such as floods, fires, and other natural and manmade disasters. Organizations that await major failures before adapting tend to enter crisis mode and find learning and response even more difficult (Staw et al., 1981; Turner, 1976). For example, following the demise of the space shuttle Challenger, NASA faced political pressures, inertia, and resource constraints that expedited some organizational changes but made other structural and cultural adjustments more difficult (McCurdy, 1993). Furthermore, in the absence of a significant environmental change or destabilizing event, lessons learned in organizations often tend to be forgotten or misapplied (de Holan and Phillips, 2004; March et al., 1991).

Even worse, because of the infrequency with which major disasters occur, trial and error organizational learning processes may lead organizational members to forget lessons from past disasters. Levitt and March (1988) argue that in the case of disaster preparedness, trial and error processes lead to "pernicious learning" – organizational leaders conclude that resources designated for disaster preparedness are left idle and should be applied elsewhere. Disaster preparation calls for a different form of learning in which organizations draw on not only their own experiences but also those of other organizations. Such network effects exist for a variety of learning processes (e.g. Argote et al., 1990; Baum and Ingram, 1998; Beckman and Haunschild, 2002).

Over the past few decades, scholars from many disciplines have advocated relational or systems approaches, as opposed to reductionist approaches that study particular events and entities in isolation (Miller, 1972; Wolf, 1980; Kastenbergh et al., 2003). Taking a relational approach will help us identify and examine learning processes as they affect and are influenced by organizations responding to major catastrophes. The issues we discuss may occur at several different levels in organizations – the interpersonal level, the sub-unit level, or the inter-organizational level.

Fortunately, we have learned a great deal about how to overcome these organizational barriers. What is needed is to instill "mindfulness" toward risks. We suggest three ways of doing so:

- Create a National Disaster Advisory Office in the White House.
- Create a Catastrophic Risk Office in Congress.
- Make FEMA into a High Reliability Organization (HRO).

13.5.4.1 Creating a National Disaster Advisory Office in the White House

No one in the White House has the job of disaster response. Yet, federal disaster response requires action by many agencies – not just FEMA but also DOD, EPA, CDC, and others. White House coordination of these executive branch activities is crucial. Just as the White House has a National Security Advisor, it needs to have an official charged with national disaster oversight. This official would also be in charge of monitoring organizational problems in the line agencies in charge of disaster response. Moreover, a natural part of the official's portfolio would be disaster prevention efforts, where the aim should be to avoid ever again being taken unawares by a "predictable surprise" like Katrina.

13.5.4.2 Creating a Catastrophic Risk Office in Congress

An integrated approach to catastrophic risk is lacking. One lesson from Katrina is that disasters are not just engineering failures, they are social system failures and failures of government. Societal and physical infrastructures can collapse. Consequently, disaster prevention cannot be considered in isolation from disaster response, mechanisms for compensation and risk spreading, and reconstruction planning. All of these issues are tightly coupled, yet the linkages receive little attention.

Under the Constitution, Congress bears the primary responsibility for developing national policy and setting national priorities. Congress authorizes and controls FEMA, the Army Corps, flood control projects, the flood insurance program, and other aspects of our nation's response to catastrophic risks. Yet Congress lacks the expertise needed to accomplish these tasks in a systematic way.

13.5.4.3 Making FEMA an HRO

Some organizations cannot afford to fail (Appendix F). Accidents can be disastrous on nuclear submarines, aircraft carriers, in air traffic control, and in hospital emergency rooms. Successful organizations of these kinds have learned to attain high reliability. By studying these organizations, experts have learned the ingredients to creating a High Reliability Organization (HRO). And there is a growing body of research on high reliability organizations (Weick 1987; Roberts 1990; Madsen et al., in press) and on high reliability systems of organizations (Roberts and Grabowski, in press; Roberts et al. 2005). Until organizations representing various aspects of disaster preparedness and disaster management seriously see themselves as systems of organizations, they cannot adequately address the problems they face.

13.6 Recommendations – Organizing for Success

The primary requirement for reconstitution of a Technology Delivery System that can and will provide an adequate and acceptable NOFDS is mobilization of the 'will' to provide such a system. If the United States decides that the catastrophe of Katrina will not be repeated, then the necessary leadership, organization, management, resources, and public support must be mobilized to assure such an outcome. One of the primary challenges is time; the clock is ticking until this area of the United States is again confronted with a severe challenge of flooding.

Recommendation 1: Seriously consider defining risk within the framework of federal, state, and local government responsibilities to protect their citizens.

Recommendation 2: Exploit the major and unprecedented role that exists for citizens, who should be considered part of governance in the spirit that those who govern do so at the

informed consent of the governed. This is the population exposed to catastrophic risks, and the people that will be protected by the NOFDS. Authorities for catastrophic risk management should ensure that those vulnerable have sufficient and timely information regarding their condition and a reciprocal ability to respond to requests for their informed consent especially regarding tradeoffs of safety for cost. The public protected by the NOFDS need to be encouraged to actively and intelligently interact with its development.

Recommendation 3: Intensify, focus, and fund Corps of Engineers modernization efforts; increasing in-house engineering capabilities and project performance, increasing in-house research and development capabilities, increasing in-house engineering performance of technically challenging projects, developing an organizational culture of high reliability founded on existing cultural values of Duty, Honor, Country, and developing a leadership role and responsibility for technical and management oversight of all phases of development of a NOFDS. Technical superiority must be re-established. Outsourcing must be balanced with in-sourcing to encourage development and maintenance of superior technical leadership and capabilities. This will require close and continuous collaboration of federal legislative, executive, and judicial agencies. This will require that the Corps of Engineers re-conceptualize itself as a pivotal part of a modular organization developing partnerships with other federal agencies, state and local governments, enterprise interests, and private stake holders.

Recommendation 4: Restructure federal/state relationships in flood control. One possible model is what has been called “modularity” -- a concept which involves provisional and functional rearrangement of units in terms of alternative configurations of tools, structures and relationships. Enhancing cooperation and collaboration, reducing confusion as to overlapping areas of operation and responsibility, and mutually supportive cross-checks and communication should all be advanced.

Recommendation 5: Develop a National Flood Defense Authority (NFDA) charged with oversight over the design, construction, operation and maintenance of flood control systems. Each state would have an equivalent organization that could foster cooperation and developments between and within the states. The Corps of Engineers, state flood control authorities, and technical advisory boards would work with the NFDA to foster application of the best available technology and help coordinate development and maintenance efforts and planning. In cooperative developments, federal and state governments would provide reliable and sustainable funding for the life-cycle of specific flood defense systems. This development should be accompanied by development of an integrated and coherent Louisiana Flood Defense Authority representing state, regional, local, city, and public stakeholders that can focus and prioritize stakeholder interests and requirements and collaborate with the Corps of Engineers in development of a NOFDS.

Recommendation 6: Because of the importance of emergency response in the NOFDS, FEMA should be developed as a high reliability organization (HRO) and returned by the executive branch to Cabinet level status. A new Council for Catastrophic Risk Management should be appointed within the White House and given oversight of disaster preparation and response. A similar body should be appointed within Congress. Incentives must be created to encourage all levels of government to deal proactively and effectively with potential national, regional, and local catastrophes.

13.7 References

- Argote, L., et al. (1990). "The Persistence and Transfer of Learning in Industrial Settings." *Management Science*, 36(2), 140-154.
- Baum, J. A. C., and Ingram, P. (1998). "Survival-Enhancing Learning in the Manhattan Hotel Industry, 1898-1980." *Management Science*, 44(7), 996-1016.
- Beckman, C. M., and Haunschild, P. R. (2002). "Network Learning: The Effects of Partners' Heterogeneity of Experience on Corporate Acquisitions." *Administrative Science Quarterly*, 47(1), 92-124.
- Bierly, P.E., and Spender, J.C. (1995). "Culture and high reliability organizations: The case of the nuclear submarine." *Journal of Management*, 21, 639-656.
- Bradbury, H., and Lichtenstein, B.M.B. (2000). "Relationality in Organizational Research: Exploring the Space Between." *Organization Science*, 11, 551-564.
- Buber, M. (1970). *I and Thou trans.* Walter Kaufman, New York: Scribner's Sons
- Clarke, L. (1999) *Mission Impossible: Using Fantasy Documents to Tame Disaster*, University of Chicago Press, Chicago, IL.
- Collins, S. (2006). *Opening Statement, Hurricane Katrina: Recommendations for Reform*, Committee on Homeland Security and Government Affairs, Washington DC, 03/08/2006.
- Columbia Accident Investigation Report (2003). *Columbia Accident Investigation Report*, U.S. Government Printing Office, 1, Washington DC.
- De Holan, P., and Phillips, N. (2004) "Remembrance of Things Past? The Dynamics of Organizational Forgetting." *Management Science*, 50, 1603-1613.
- Dekker, S. (2002) *The Field Guide to Human Error Investigations*, Ashgate, Aldershot, England.
- Department of Homeland Security (2006). "Our Top Five Most Frequently Asked Questions." *NIMS Alert*, No. 04-006, Washington DC..
- Department of Homeland Security (2004a). *National Response Plan*, U.S. Government Printing Office, Washington DC.
- Department of Homeland Security (2004b). *National Incident Management System*, U.S. Government Printing Office, Washington DC.
- Freeman, J. and Farber, D. (2005). "Modular environmental regulation." *Duke Law Journal*, 54, 795-912.
- Grabowski, M., and Roberts, K. H. (1999). "Risk Mitigation in Virtual Organizations." *Organization Science*, 10, 704-721.
ALSO IN *JOURNAL OF COMPUTER MEDIATED COMMUNICATION*, 1998, 3, 4.
- Heath, C., and Staudenmayer, N. (2000). "Coordination Neglect: How Lay Theories of Organizing Complicate Coordination in Organizations." *Research in Organizational Behavior*, B.M. Staw and R.I. Sutton (Eds.), Elsevier, New York, 22, 153-191.
- Houck, O.A. (2006). "Can We Save New Orleans?" *Tulane Environmental Law Journal*, 19(1), <<http://www.law.tulane.edu/tuexp/journals>> (Apr. 25, 2006).

- Kastenberg, W.E., Hauser-Kastenberg, G., and Norris, D. (2003). "Shifting the Paradigm of Risk Analysis." *Proceedings of the International Seminar on Nuclear War and Planetary Emergencies*, 29th Session, Erice, Italy, May 10-15.
- Lakoff, A. (2006). "From Disaster to Catastrophe: The Limits of Preparedness." *Understanding Katrina: Perspectives from the Social Sciences*, The Social Science Research Council, <<http://understandingkatrina.ssrc.org/Lakoff>> (May 15, 2006).
- Levitt, B., and March, J. G. (1988). "Organizational Learning." *Annual Review of Sociology*, 14: 319-340.
- Madsen, P., Desai, V., and Roberts, K. H. (in press). "Designing for High Reliability: The Birth and Evolution of a Pediatric Intensive Care Unit Organization." *Science*
- March, J. G., Sproull, L., and Tamuz, M. (1991). "Learning from Samples of One or Fewer. Organization." *Science*, 2, 1-13.
- McCurdy, H. E. (1993). *Inside NASA: High Technology and Organizational Change in the U.S. Space Program*, Johns Hopkins Press, Baltimore MD.
- Miller, J.G. (1972). *Living Systems*, McGraw Hill, New York.
- National Research Council (2002). *Review Procedures for Water Resources Project Planning*. National Academies Press., Washington DC.
- Nelson, R. R., and Winter, S. G. (1982). *An Evolutionary Theory of Economic Change*, Cambridge MA.
- Perrow, C. (2005). *Using Organizations: The Case of FEMA*, Social Science Research Council Forum, <<http://understandingkatrina.ssrc.org>> (Apr. 21, 2006).
- Roberts, K. H. (1990) "Some Characteristics of One Type of High Reliability Organization." *Organization Science*, 1, 160-176.
- Roberts, K. H., and Bea, R. G. (2001a). "Must Accidents Happen: Lessons from High Reliability Organizations." *Academy of Management Executive*, 15, 70-79.
- Roberts, K. H., and Bea, R. G. (2001b). "When Systems Fail." *Organizational Dynamics*, 29, 179-191.
- Roberts, K. H., Madsen, P. M., and Desai, V. M. (2005). "The Space Between in Space Transportation: A Relational Analysis of the Failure of STS 107." *Organization at the Limit: Lessons from the Columbia Disaster*, W. H. Starbuck and M. Farjoun (Eds.), Blackwell, Malden MA, 81-98.
- Roberts, K. H., and Wong, D. (2006). "Outsourcing and Temporary Workers." *Encyclopedia of Industrial/Organizational Psychology*, S. Rogelberg (Ed.), Sage, London, England.
- Roberts, K. H., and Grabowski, M. (in press). "Risk Mitigation in Healthcare Organizations and in Aggregations of those Organizations." *Human Error in Medicine*, M.S. Bogner (Ed.), 2nd Ed., Erlbaum, Hillsdale NJ.
- Senate Committee on Homeland Security and Governmental Affairs (2006). *Hurricane Katrina: A Nation Still Unprepared*, Washington DC.
- Staw, B. M., Sandelands, L. E., and Dutton, J. E. (1981). "Threat-Rigidity Effects in Organizational-Behavior - a Multilevel Analysis." *Administrative Science Quarterly*, 26(4), 501-524.

- Tate, B., and Halford, G. (2002). "New moves will streamline gov't." *Engineer Update*, U.S. Army Corps of Engineers, 26(7), <<http://www.hq.usace.army.mil/cepa/pubs/jul02/story3.htm>> (Mar. 20, 2006).
- Turner, B. A. (1976). *Organizational and Interorganizational Development of Disasters*. Wykeham Publishers, London, United Kingdom.
- United States Army Corps of Engineers (2002). "USACE Strategic Workforce Planning." *USACE Command Brief*, <<http://www7.nationalacademies.org/ffc/usace.pdf>> (Mar. 30, 2006).
- Vaughan, D. (2005). "Systems Effects: On Slippery Slopes, Repeating Negative Patterns and Learning from Mistakes." *Organization at the Limit: Lessons from the Columbia Disaster*, W. H. Starbuck and M. Farjoun (Eds.), Blackwell Publishing, Malden MA, 41-59.
- Weick, K. E. (1987). "Organizational Culture as a Source of High Reliability." *California Management Review*, Winter, University of California, Berkeley CA.
- Weick, K. E., and Sutcliffe, K. (2001). *Managing the Unexpected*, Jossey Bass, San Francisco CA.
- Wolf, F. A. (1980). *Taking the Quantum Leap*, Harper & Row, New York.
- Working Group for Post-Hurricane Planning for the Louisiana Coast (2006). *A New Framework for Planning the Future of Coastal Louisiana after the Hurricanes of 2005*, University of Maryland Center for Environmental Science, <<http://www.umces.edu/la-restore>> (Apr. 23, 2006).

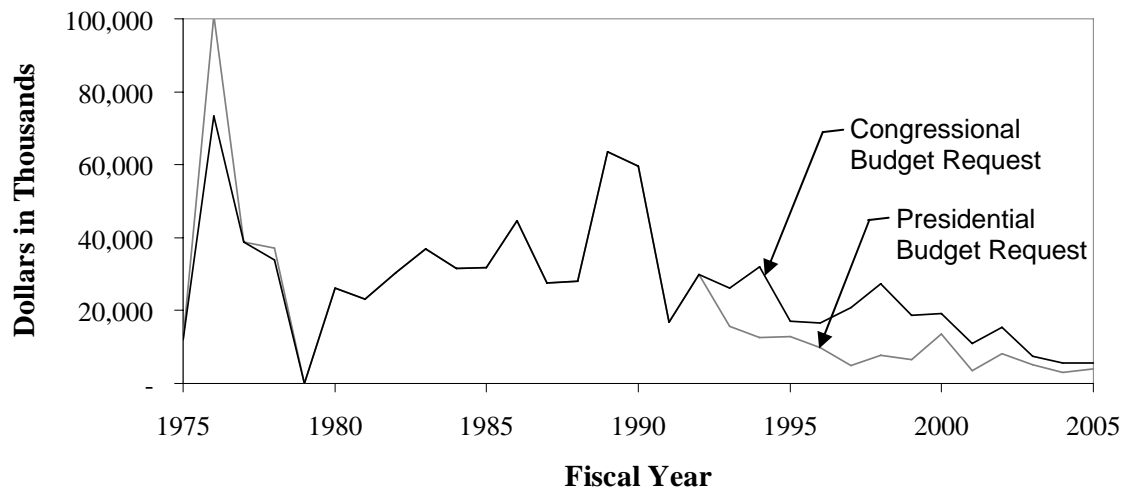


Figure 13.1: Lake Pontchartrain and Vicinity Project Construction Appropriations Over the Past 30 years [2005 dollars]; President's Budget Request (grey) and the Amount Recommended by Congress (black).



Figure 13.2: Artwork by Jan Fitzgerald illustrating the debate surrounding President Bush's initiative to streamline the federal government (Tate and Halford 2002).

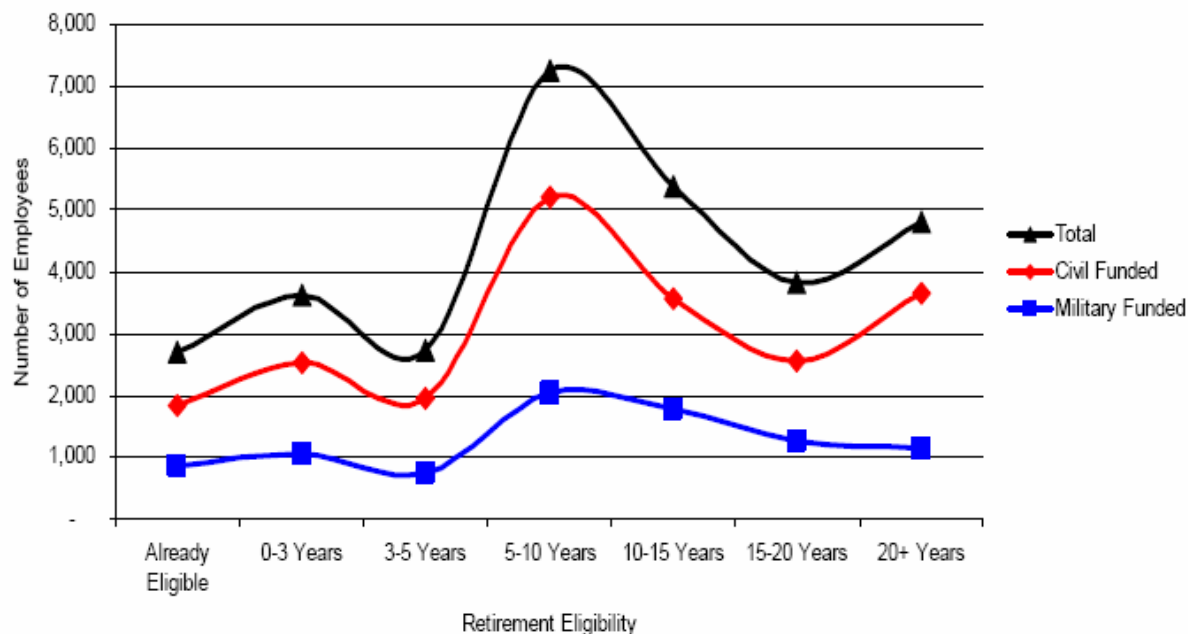


Figure 13.3: Human Capital Planning Projected Retirement (USACE 2002).

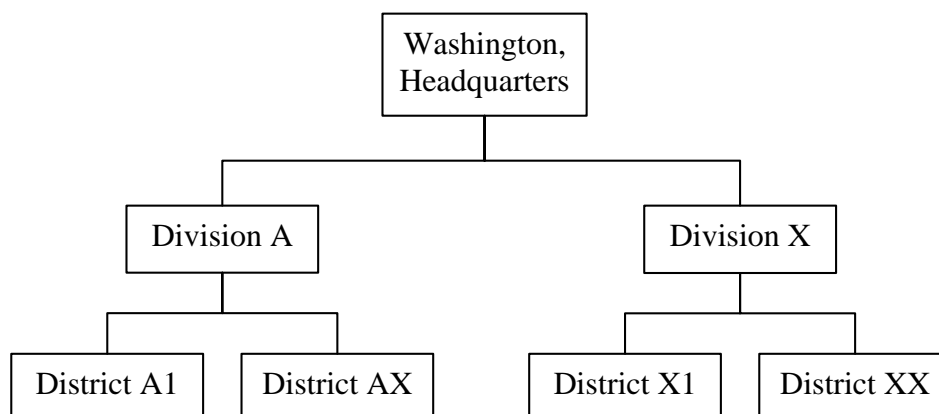


Figure 13.4: Conceptual Organizational Chart of the U.S. Army Corps of Engineers Civil Works Program.¹

¹ The Corps Civil Works Program is composed of 8 Divisions and 38 subordinate districts. Prime Power, ERDC, Centers, and FOAs are not shown for clarity. In addition, a 9th provisional division with four districts was activated January 25, 2004, to oversee operations in Iraq and Afghanistan. A more complete organizational chart is available in *USACE 2012 – Appendix G, Resource Analysis*, page 1.

CHAPTER FOURTEEN: ENGINEERING FOR SUCCESS

The tragedies of Hurricanes Katrina and Rita in 2005 have revealed to the world the enormous challenge Louisiana now faces. South Louisiana appears to have entered a period when the convergence of two powerful forces is working against its survival. Since the 1950's, the processes driving coastal loss have continued only slightly abated. Since 1990, meteorological and oceanic processes driving tropical systems have more frequently generated category 4 and 5 hurricanes. More destructive hurricanes are predicted for coming decades. ~ South Louisiana's ongoing peril is the continued overlap of weakened hurricane protection with more frequent and intense hurricanes.

In light of this predicament, how can the coast and culture of south Louisiana survive? The survival of a culture and a region is at stake. Hurricanes Katrina and Rita may have narrowed the field of discussion from what we might want, down to what we absolutely need. There is a growing consensus that what is needed is a pragmatic and effective strategy to integrate both coastal habitat restoration and engineered flood protection, such as levees. This strategy must be established soon and while under duress.

John Lopez (2006).

The Multiple Lines of Defense Strategy to Sustain Louisiana's Coast
Report to Lake Pontchartrain Basin Foundation, New Orleans.

14.1 Introduction

At the present time, the federal government is just completing a significant effort to re-establish the New Orleans Flood Defense System (NOFDS) to “pre-Katrina conditions” by a target date of June 1, 2006. The federal government has proposed to further improve the NOFDS to meet “100-year flood conditions” by 2010. Studies are currently underway by the Corps of Engineers to define an expanded and more reliable NOFDS (see Appendix G). In this Chapter we explore options for the engineering elements that could be provided in an improved long-term NOFDS.

The first question to be addressed in going forward is: “what should we do about providing adequate flood protection for the greater New Orleans area?” To the people who lived and continue to live in this area, this is not a question. These people are in the process of rebuilding their homes and lives. A majority of people who live in this area are committed to rebuilding and continuing the development of this area. Some have and will decide not to return; they will rebuild elsewhere.

The real question is about the ‘we’. The following thoughts on this question were advanced by former Speaker Newt Gingrich (2006):

Shortly after Hurricane Katrina devastated New Orleans, Speaker of the House Dennis Hastert wondered aloud whether the Federal Government should help rebuild a city much of which lies below sea level. The most tough-minded answer to that question demonstrates that rebuilding and protecting New Orleans is in

the national interest. Reason: The very same geological forces that created that port are what make it vulnerable to Category 5 hurricanes and also what make it indispensable.

If engineering the Mississippi made New Orleans vulnerable, it also created enormous value. New Orleans is the busiest port in the U.S.; 20% of all U.S. exports and 60% of our grain exports, pass through it. Offshore Louisiana oil and gas wells supply 20% of domestic oil production. But to service that industry, canals and pipelines were dug through the land, greatly accelerating the washing away of coastal Louisiana. The state's land loss now totals 1,900 sq. mi. that land once protected the entire region from hurricanes by acting as a sponge to soak up storm surges. If nothing is done, in the foreseeable future an additional 700 sq. mi. will disappear, putting at risk port facilities and all the energy-producing infrastructure in the Gulf.

...Washington also has a moral burden. It was the Federal Government's responsibility to build levees that worked, and its failure to do so ultimately led to New Orleans' being flooded. The White House recognized that responsibility when it proposed an additional \$4.2 billion for housing in new Orleans, but the first priority remains flood control. Without it, individuals will hesitate to rebuild, and lenders will decline to invest.

How should flood control be paid for? States get 50% of the tax revenues paid to the Federal Government from oil and gas produced on federally owned land. States justify that by arguing that the energy production puts strains on their infrastructure and environment. Louisiana gets no share of the tax revenue from the oil and gas production on the outer continental shelf. Yet that production puts an infinitely greater burden on it than energy production [from] other federal territory puts on any other state. If we treat Louisiana the same as other states and give it the same share of tax revenue that other states receive, it will need no other help from the government to protect itself. Every day's delay makes it harder to rebuild the city. It is time to act. It is well past time.

For us it is not a question of *if* we go forward to provide an adequate and acceptable NOFDS. It is a question of *how* we go forward. Going forward will demand a lot of all involved including vision, commitment, responsibility, respect, organization, cooperation, leadership, knowledge, resources, preparations, time, and some good luck. While this Chapter examines the engineering aspects of providing long-term hurricane flood protection for the greater New Orleans area, it should be clearly understood that a PREREQUISITE to a successful venture must be re-engineering the Technology Delivery System (see Chapter 13) needed to develop such a system. History has clearly shown that without an effective and sustainable TDS, we can expect a deficient and defective long-term NOFDS. History will repeat itself if we let it.

During the next several decades, hurricane seasons are expected to produce greater numbers of more severe storms. Unnecessary delays in embarking on development and realization of a long-term NOFDS only increase our chances of failing. We learned this lesson during the 40-year period between the disastrous flooding of New Orleans in 1965 (hurricane Betsy) and the catastrophic flooding of 2005 (hurricane Katrina). Now is the time for careful and deliberate thought followed by effective and timely action. Another disastrous flooding of the greater New Orleans area should not be an option.

14.2 Engineering Considerations

The ILIT addressed two key aspects associated with the engineering considerations of going forward: (1) the NOFDS physical facilities, and (2) the engineering criteria and guidelines for these facilities.

14.2.1 Physical Facilities

Evaluation of the options for NOFDS physical facilities requires a basic understanding of the natural environmental - geological - ecological setting of this area, the commercial - industrial complex established in this area, and unique cultural - social - institutional - political elements. This is a very complex system whose future is shadowed by its past.

A systematic and integrated study needs to be performed of the options for provision of physical facilities so that informed choices can be made about how best to provide long-term flood protection for the greater New Orleans area. The NOFDS is part of an even larger challenge that involves other parts of the Gulf coast and the floodplain of the Mississippi River (Dean, 2006). The real threats of increased hurricane activity and intensity, coastal degradation, subsidence, and climate change (rise in sea level, increase in rainfall and flood potential) must be recognized and appropriate and effective preparations put in place to help protect life and property in this area.

The Mississippi River and the Gulf of Mexico have been interacting in this part of the United States for millions of years (Kelman, 2003). As a result of sediments transported and deposited by the Mississippi River during the past 100,000 years, a vast complex of delta lobes have developed where a succession of different river channels meet the Gulf of Mexico (Coleman, 1988). Sixteen of these lobes have been developed and abandoned during the past 20,000 years. The sediments deposited by these delta lobes dominate the geology of this area, and the recently deposited sediments reach thicknesses exceeding 500 feet (U.S. Army Corps of Engineers, 2004).

The Mississippi Delta is a broad wedge-shaped floodplain whose top is about where the Atchafalaya River branches off from the Mississippi River and whose broad curved base is the Gulf of Mexico coastline (about 150 miles wide) (Sparks, 2006). The coastline is delineated with a long line of barrier islands. The shape of this delta is determined by sediment accumulation, compaction, subsidence, growth faulting, changing sea level, and most recently by man's activities. Recent information indicates that since the sea reached its present level (about 6,000 years ago), six major lobes including a developing new one at the mouth of the Atchafalaya River have existed. The modern Birdsfoot Delta that lies to the southeast of New Orleans (Plaquemines parish) has existed for only about the last 1,000 years.

The river has been trying to change its course to the Atchafalaya River (100 miles to the west) as the length of the Mississippi River to the Gulf of Mexico has increased (now more than 200 miles). In order to maintain New Orleans as a deepwater port in the 1950s, the Corps of Engineers constructed the Old River Control Structure to help divert about 30% of the Mississippi River water down the Atchafalaya and keep the remainder flowing to the Gulf through its present course. In 1973, a flood on the Mississippi River almost caused failure of the Old River control Structure. The Corps completed a new auxiliary structure in 1985 to take some of the pressure off the Old River control Structure. At the present time, the Atchafalaya lobe is actively building toward the Gulf of Mexico and the lobe south of New Orleans is regressing.

A variety of processes have altered the natural process of land building by the Mississippi River and its tributaries (Hallowell, 2005; Committee on the Restoration and Protection of Coastal Louisiana, 2006; Zinn, 2004, 2005a, 2005b). These include the building of upstream dams and flood control structures (decreased sediment supply), building of levees (which do not permit sediment transport to adjacent areas), building of canals and pipelines (oil and gas exploration and production), building of navigable waterways (e.g.: Gulf Inter-Coastal Water Way, Mississippi River Gulf Outlet), and configuration of the current mouth of the Mississippi river to “shoot” sediments out into the Gulf of Mexico, over the edge of the continental shelf, in order to reduce the need for active dredging to maintain navigability of the main river channel for shipping. All have had their effects on reducing replenishment of sediments to keep up with subsidence, on the balance coastal transport processes, and on providing nutrients to sustain freshwater wetlands.

With population and industrial growth along the Mississippi River and its tributaries (it drains about 40% the United States), influx of byproducts and waste products have also taken their toll on the wetlands. Exploration for and production (extraction, transport) of hydrocarbons have also taken their toll on wetlands and contributed to land loss. The rise of sea level has also taken its toll. The result is a rapidly degrading and regressing coastline. This coastline is projected to loose about 10 square miles of land per year during the next 50 years (Dean, 2006; Sparks, 2006). The rapidly regressing coastline has had important effects with regard to the increase in hurricane risk affecting the NOFDS.

The NOFDS is faced not only with the challenges associated with potential hurricane surges and waves, but also with potential floods from the Mississippi River, with subsidence and compaction, with reduction of the storm-buffering provided by coastal barrier islands and wetlands, and with potential water and saltwater ingress provided by man-made waterways. Oliver Houck (2006) addressed these challenges:

So here is the starting point: exactly what we do want the Louisiana coast to look like, to do for us, for say, the next century? ...Earth to Louisianans: you really can't have this cake and eat it too. With all due respect, it is not just a matter of doing everything we want 'smarter.' It is a matter of getting straight what we want, and that comes first. What comes next is the hardest step for any American community to take, and shall be heresy in South Louisiana. A plan. The mere mention of planning raises blood pressures and brings on cries of Godless Communism. What we have had in the city of New Orleans and along the entire gulf coast is planning by default (local attorney Bill Borah calls it 'planning by surprise'). Planning takes place. It's just that we haven't taken part in it. Where water resources are concerned, it starts with real estate developers, port authorities, levee boards and other outside-the-ballot-box enterprises, their projects facilitated and funded by the Army Corps of Engineers. In their minds, the only question is a technical one: what kind of engineering do we need to get our project done? The system has produced the expected results: more rip-rap here, more drainage there, and levees to the horizon. The goal is - although it is never stated anywhere - to develop as much of the coast as possible. When you add the projects up, they determine the destiny of the city and South Louisiana.

What is apparent is that these levees, designed by engineers and approved by Congress, are the basic planning documents for the future of South Louisiana. What is north of these levees will be developed. What is south of them will be

anyone's guess, although not for long; the map on global warming shows these coastal marshes gone within a century. De facto, we end up with a wall. Not all that adequate a wall, by the way. Only Category three, if that. Can you imagine the costs of maintaining even a Category three levee system winding back and forth to the Gulf from New Orleans to Texas? Can we imagine what will happen when development piles in behind it, and then gets flooded? Do we already know, from Lakeview and New Orleans East, what happens to land elevations behind levees once they are drained and paved?

Our choice is to start this process from the other end. If we do, another range of options open. There are a dozen major towns across the southern tier with thousands of homes and residents, and they deserve protection. But the way to provide it may be with the same kind of ring levee systems that protects (or should) New Orleans and its surrounding parishes, supplemented by flood gates at the mouths of the main canals. Or, it may mean peninsular levee systems down the historic ridges of the bayous, protecting what has always been the high ground. ...Problem is, we have lacked the process - we have lacked even the language - for such a discussion. In addition to scientists and engineers, we may need some social workers. In saying this, I am most serious.

The ILIT examined two basic alternatives to develop a long-term NOFDS. The first was constructing levees, floodwalls, and pump stations capable of providing a long term NOFDS. At the present time, efforts are underway to provide "100-year" flood protection. But, the question is why "100-year" protection? Why not 1,000 year or 10,000 year protection (frequently posed as Category 4 or 5 hurricane protection)?

Our studies of economic cost-benefit guidelines, and historic and current standards of practice for public facilities in the United States and elsewhere indicated that protection against disastrous flooding of the greater New Orleans area should be for conditions having average return periods much more demanding than the present goal of "100-year" flood protection. This issue was addressed by another very similar region that must defend its population and commercial enterprises at elevations up to 23 feet below sea level - the Netherlands (Netherlands Water Partnership, 2005):

Our standards are accepted risks related to the design-criteria of our dikes. Those standards are laid down in the Flood Defense Act. For the economically most important and densely populated part of the country, we design our dikes and dunes to be strong enough to withstand a storm-situation with a probability of 1 to 10,000 a year. That means, that a Dutchman - if he should live a 100 years - has a chance of 1 percent to witness such an event. For our parliament, these odds became the acceptable standard. For the less important coastal areas we calculate the probability of 1 to 4,000 and along the main rivers 1 to 1,250.

This background was developed largely after the Netherlands suffered catastrophic flooding of the country in 1953. This flooding was comparable to the flooding of the greater New Orleans area in the wake of hurricane Katrina (approximately 1,800 dead, 50,000 destroyed homes, 350,000 acres of flooded land). It was also preceded by a history that included a large number of malfunctions that included poor organization, bad maintenance, warnings not heeded, poor communications, underestimation of the danger, negligence, lack of preparedness (Jurjen

Battjes, personal communication; Dec. 30, 2005). This same history was repeated in the catastrophic failure of the NOFDS.

Following the 1953 catastrophe, the Dutch vowed “never again” and developed a system that is today a model of advanced engineering and water resource management. It also provides a model for the organizational re-engineering required to realize the system they have in place today, and that they continue to maintain and improve. This organization is a centralized Rijkswaterstaat which is the national public works department in charge of all flood defense works. This department has direct ties and interfaces with the local agencies responsible for continued development, maintenance, and improvement of flood defense work (including evacuation and disaster recovery). However, the Dutch have learned the sad lessons of trying to overwhelm nature with engineered works. They have seen many unintentional consequences from such an approach surface as very severe negative environmental and quality of life impacts. And, they learned from these mistakes and gone on to remediate the mistakes and develop new strategies (Netherlands Water Partnership, 2005):

Climate changes are increasing the likelihood of flooding and water-related problems. In addition population density continues to increase, as does the potential for economic growth, and consequently, the vulnerability to economic and social disaster. Two undesirable developments that, in terms of safety, exacerbate one another - a grown risk with even larger consequences. As such, the safety risk is growing at an accelerated pace (safety risk - chance multiplied by consequence).

The Netherlands is changing its approach to water. This change involves the idea that the Netherlands will have to make more frequent concessions. We will have to relinquish open space to water, and not take back existing open spaces, in order to curb the growing risk of disaster due to flooding. We will also need to limit water-related problems and be able to store water for expected periods of drought. By this we do not mean space in terms of the height of ever taller levees or depth through continued channel dredging, but space in the sense of flood plains. This approach will require more area, but in return we will increase our safety and limit water related problems. Safety is an aspect that must play a different role in spatial planning. Only by relinquishing our space can we set things right; if this is not done in a timely manner, water will sooner or later reclaim the space on its own, perhaps [in a] dramatic manner.

The Dutch continue to be challenged by their countrymen not to become conceited or complacent - they are devoted to a culture of continuous improvements in their flood protection.

Our consideration of this background indicated that the most attractive option for provision of an acceptable and sustainable long-term NOFDS is *one of re-establishing and enhancing selected natural defenses supplemented with engineered works as necessary to provide long-term flood protection*. Guidelines and many useful insights are provided by John Lopez (2006) in the report *The Multiple Lines of Defense Strategy to Sustain Louisiana's Coast* about how such an option might be developed. Additional background for development of this option is also provided in the reports *Coast 2050: Toward a Sustainable Coastal Louisiana* (Louisiana Coastal Wetlands Conservation and Restoration Task Force 1998), *Ecosystem Restoration Study* (U.S. Army Corps of Engineers 2004), *Drawing Louisiana's New Map* (National Research Council 2006), and *A New Framework for Planning the Future of Coastal*

Louisiana after the Hurricanes of 2005 (Working Group for Post-Hurricane Planning for the Louisiana Coast 2006). Results contained in these studies provide a coherent and substantial basis for development of a long-term NOFDS. Lopez (2006) proposes eleven *Lines of Defense* (see Figure 14.1):

1st Offshore shelf within the Gulf of Mexico: *The offshore shelf ranges in depth from 300 feet at the shelf edge to zero depth at the gulf shoreline. Its width varies from a few miles to hundreds of miles. The primary benefit of the shallow shelf is to dramatically reduce wave height and wave energy from an approaching tropical system. A negative aspect of the shelf is that it will promote higher storm surges inland. The variable influences storm surges due to the geometry of the shelf needs to be considered for storm surge analysis. Also, dredging activities on the shelf should avoid increasing shoreline erosion by wave refraction around dredge holes. The gulf fisheries and the oil and gas industry are key economic aspects of the shelf. Examples: Narrow shelf at the mouth of Mississippi River & Wide shelf offshore from Cameron Parish*

2nd Barrier Islands: *The Louisiana barrier island shoreline is characterized by fragmented barriers or shoals with low vertical profiles and low sand content. However, barrier islands provide an important wave barrier for interior sounds and coastal marsh. The primary benefits of barrier islands are the near-complete reduction in wave height and the slight reduction in storm surge further inland. A negative aspect of barrier islands is their ephemeral nature and unpredictable local impacts to them from hurricanes. Barrier islands also have significant recreational aspects such as fishing and birding. Examples: Chandeleur Islands and Grand Isle*

3rd Sounds: *The primary benefit of the sounds is to provide a relatively shallow water buffer to deep water currents. Sounds do have a negative aspect during storms by allowing waves to re-generate on the sound side of barrier islands. Also, sounds may cause storm surge and wave erosion on the back side of barrier islands.*

4th Marsh Landbridges: *Marsh landbridges are areas of emergent marsh with relative continuity compared to adjacent bays, sounds or areas of significant marsh/land loss. Ideally, landbridges connect other elevated landforms such as natural ridges. Since some ridges are developed and have adjacent levees, marsh landbridges may also bridge adjacent levee systems and economic corridors. Marsh landbridges compose much of the residual internal framework of the coast which reduces fetch and shoreline erosion of interior marshes and lagoons. Landbridges impede storm surge movement inland and protect other emergent marsh areas that may perform the same function. Some landbridges are threatened themselves by various processes of marsh loss and need to be sustained through restoration and maintenance. The landbridges represent an increasing fraction of the remaining emergent marsh of the coast and provide typical high productivity and fishery benefits typical of coastal wetlands. Examples: East Orleans landbridge, Biloxi Marsh landbridge, Barataria Basin landbridge, Upper Terrebonne Bay landbridge, Grand Lake-White Lake landbridge, Western Marsh Island landbridge, south Calcasieu Lake landbridge*

5th Natural Ridges: *In southeast and central Louisiana, most natural ridges are the natural levees of abandoned distributary channels. These channels now act as tidal channels and are often colloquially named bayous or rivers. In southwest Louisiana, most natural ridges are cheniers running parallel to the Gulf coastline. Natural ridges may have continuous elevation of several feet and, therefore, will impede overland flow across the ridge and potentially reduce storm surge. Natural ridges often define (at least historically) the hydrologic basins of the coast. Natural ridges are most effective when they have at least 6 feet of elevation and well drained soils to maintain upland forests. Forests will also slow the movement of overland flow and may also provide a wind barrier. Natural ridges tend to be the economic corridors across the coast including primary state highways and coastal communities. These highways are also likely to be evacuation routes. Examples: Bayou la Loutre, Bayou Lafourche*

6th Manmade Soil Foundations: *Manmade soil foundations for transportation may provide incidental benefit to storm surges. Railroads, highways and spoil banks may run parallel to the coast and locally provide a manmade ridge several feet [high]. These foundations may have settled and may need improvement to provide reliable transportation routes without chronic flooding. If highway improvements are contemplated, the effects on storm surge may be considered. Examples: Highway 90, Hwy 82*

7th Flood Gates: *Flood gates are typically designed to withhold flood water and, therefore, remain open under most conditions. Flood gates are generally open so as not to impede navigation or natural ebb and flow of tides and aquatic organisms. Flood gates would be closed during a threat of flooding and to reduce flood tides in channels. Because of the generally low elevation of the coast, the effectiveness of flood gates may depend on the nearby topography or constructed features such as levees or spoil banks. Examples: Bayou Bienvenue, Bayou Dupre*

8th Flood Protection Levees: *Flood protection levees are designed and constructed for flood protection of municipalities or other coastal infrastructure features. Levees are generally designed to be an absolute barrier defining a flood side and a protected side. The intent is to have zero storm surge flooding on the protected side, but an unintended consequence may be to increase water levels on the flood side. Levees are generally not designed to be overtopped or to withstand significant wave erosion. Exceptions include “potato levees” or other low relief levees designed to reduce flooding from non-storm tides. Typical hurricane protection levees protect limited portions of the coast with intense economic development. Examples: St. Bernard levee, Jefferson and Orleans Parish levees on Lake Pontchartrain*

9th Flood protection pumping: *Pumping stations are generally within leveed areas and are used to reduce flood risk from rainfall and are not designed to pump out flood water in the case of a levee breach. Most pumping stations are not prepared with fuel, staff or other requirements to be effective to pump out flood water from a significant levee breach. Generally, these are large capacity pumps which displace water vertically above the water level on the flood side of the levee. Pumping stations are generally to protect areas of intense development. Examples: Orleans and Jefferson Parish’s pumping stations.*

10th: Elevated Homes and Businesses: All homes and businesses in south Louisiana are subject to being flooded if they are not elevated above the normal land elevation. Even those behind levees are not 100% safe. Hurricanes Katrina and Rita made this painfully clear. All attempts to reduce storm surge height or its extent are limited by the intensity and attributes of particular storm events. Since there will always be the potential of a storm exceeding the limits of protection from storm surges, immovable assets such as homes and businesses should be elevated to the appropriate flood elevation risk. This is the last line of defense for immovable assets. Elevated homes also provide important side benefits such as improved protection from termites and more economic capacity to re-level or raise the houses due to settlement or increased flood risk. Example: pre-1940 housing in New Orleans, LUMCON, Marina del Ray in Madisonville

11th Evacuation: Evacuation routes are typically highways, but could also include other means of transportation such as railroads, air transportation, etc. Evacuation routes are the last line of defense for people or moveable assets. Evacuation routes and procedures should be established for the coast. Ideally, evacuation routes may also serve as re-entry routes for first responders and as routes to re-populate after a storm event. Evacuation routes are generally selected based capacity to move a large number of people to safer areas as a storm approaches the coast. Some routes may be subject to flooding quickly and need to be improved. Examples: Regional contra-flow evacuation plan for southeast Louisiana.

The Corps of Engineers and other organizations are continuing work to develop an advanced and more reliable NOFDS that is more compatible with the natural, industrial and social environments of southern Louisiana. The Working Group for Post-Hurricane Planning for the Louisiana Coast recently concluded (2006):

In the long term, hurricane protection for larger population centers, including the New Orleans region, can only be secured with a combination of levees and a sustainable coastal landscape. This will require adapting to changing conditions by re-establishing the constructive processes associated with distributing Mississippi River water and sediments across the coastal landscape, as well as alleviating the other destructive effects of past or future human activities.

With presently observed subsidence rates and anticipated acceleration of sea-level rise, most - although not all - of the coastal landscape could be maintained through the 21st century. And with efficient management of the river's resources, this landscape could be expanded in some places. However, this result can only be achieved with very aggressive, strategic, and well-informed restoration efforts, varying in size and objective but integrated within a landscape management plan.

The challenges associated with rehabilitation and improvement of the NOFDS need to be addressed in an integrated way combining public and social, organizational and institutional, natural and environmental, and commercial and industrial considerations. This is a “systems problem” that has many parts which are interactive, interdependent, and highly adaptive. We need to understand potential impacts, positive and negative, on the parts of this system so that wise choices and informed decisions can be made on how best to proceed. This is a different kind of “engineering problem” in which the Technology Delivery System used to address that problem is of utmost importance. Gerald Galloway (2006) summarized these issues:

Since 1983, when the Water Resources Council was effectively abolished, there has been no central direction to or coordination of federal water efforts, among the many departments that deal with water issues. Congress remains locked in a turf-conscious committee system that does not encourage coordination. Except for enforcing water quality standards there is little federal guidance, other than budgetary or ad hoc initiatives, on other water issues.

Given the present policy vacuum and the reluctance on the part of Congress and the administration to support comprehensive planning, New Orleans and coastal Louisiana will have to develop, in coordination with federal agencies, their own vision for the future and move ahead in a way that brings together solutions to the many water challenges facing the region. this comprehensive plan must address all aspects of coastal Louisiana's water challenges.

Each of the alternatives for development of a long-term NOFDS has its pluses and minuses, costs and benefits. It is clear that these alternatives need to be continually examined in an integrated and systematic way. The fundamental technology exists to develop an adequate long-term NOFDS. The question is not “can we do it?” The question is “will we do it?”

14.3 Engineering Criteria and Guidelines

The basic technology exists to develop an effective and efficient NOFDS. A major challenge is timely and proper application of this technology. The following recommendations are made to facilitate such application.

Recommendation 1: Develop an integrated and coherent Flood Defense System for the greater New Orleans area (NOFDS) to provide desirable and acceptable levels of flood protection throughout its life-cycle. Particular attention must be paid to interfaces and interdependencies in this system. The NOFDS should be balanced, complete, cohesive, clear, consistent, and have controls and continuity. The NOFDS should be based on the best available and safest technology and most up-to-date legal standards. Risks should be properly identified, contained and compartmentalized. The system must recognize the unique natural environmental setting including its geology, meteorology, oceanography, the Mississippi River floodplains, deltas and wetlands, subsidence, and the rise in sea level and frequency and intensity of hurricanes. The system must also recognize and accommodate the unique societal and cultural environments of this area.

Recommendation 2: Develop a NOFDS based on enhancing natural defenses supplemented with engineered defenses that incorporate concepts of defenses in depth, robustness or resilience, and fail-safe performance. Selective re-establishment of natural coastal defenses and wetlands and restored floodplains to provide for river floods should be supplemented with engineering works that together have the capabilities of providing desirable and acceptable levels of flood protection. Coastal management must be focused on providing safety from flooding and environmental protection. Water should be given space. Some areas will have to be returned to nature and judicious and wise decisions must be reached on which areas will be populated and developed and the levels of protection that will be provided to these areas. Engineering works should include raising, strengthening and defending levees, providing floodgates and storm surge barriers, positioning and defending modern pump stations. Engineering must also address compartmentation to limit potential flooding and adequate and

effective evacuation measures to help limit effects on people and their possessions. A robust NOFDS will require a combination of appropriate configuration of engineered elements and components, ductility or an ability to deform and stretch and not lose important performance characteristics, excess capacity so that if some elements or components are overloaded or do not perform desirably, desirable protection can be maintained, and appropriate correlation or mutual relationships so that desirable protection is realized. Fail safe characteristics should be provided in all of the important elements of the NOFDS so that when the design and ultimate performance conditions are exceeded, the performance characteristics are not appreciably compromised.

Recommendation 3: Develop a NOFDS founded on advanced Risk Assessment and Management principles for all phases in the life-cycle including concept development, design, construction, operation, and maintenance. These principles should address natural, analytical modeling, human and organizational performance, and knowledge acquisition and utilization uncertainties and be based on proactive, reactive, and interactive risk assessment and management approaches. These approaches should be based on reductions in likelihoods of failure, reduction in the consequences associated with potential failures, and increases in detection and correction of developments that can lead to failures. Advanced Risk Assessment and Management approaches should be used to provide decision makers with information to define what levels of protection should be provided for which areas to be protected and how much can and should be spent for those purposes.

Recommendation 4: Develop updated engineering guidelines and procedures for all elements and components to be incorporated in the FDS for all life-cycle phases based on proven state-of-practice and state-of-art technology. Where technology gaps are identified, substantial development programs should be implemented to fill them with existing research results. Where technology gaps can not be filled with existing research results, research should be undertaken or sponsored to enable their timely filling.

Recommendation 5: Develop, implement, and enforce advanced Quality Assurance and Quality Control methods and procedures for all life-cycle phases of the NOFDS. Quality Assurance (proactive) and Quality Control (interactive) measures are of particular importance to help disclose 'predictable surprises' and variances in the desirable quality characteristics of the elements and components in the NOFDS. These methods and procedures should be used in all life-cycle phases of the NOFDS including concept development, design, construction, operation, maintenance, and continued improvement. These procedures and measures need to assure that the best available and safest technology is used and used properly.

14.4 References

- Coleman, J. M. (1988). "Dynamic Changes and Processes in the Mississippi River Delta." *Geological Society of American Bulletin*, 100 (7).
- Committee on the Restoration and Protection of Coastal Louisiana (2006). *Drawing Louisiana's New Map: Addressing Land Loss in Coastal Louisiana*, Ocean Studies Board, National Research Council, The National Academies Press, Washington, DC, <<http://www.nap.edu>> (May 1, 2006).
- Dean, R. G. (2006). "New Orleans and the Wetlands of Southern Louisiana." *The Bridge*, National Academy of Engineering, Washington DC, <<http://www.national-academies.org>> (May 1, 2006).

- Galloway, G. E. (2006). "Restoring Coastal Louisiana: Planning without a National Water Policy," *The Bridge*, National Academy of Engineering, Washington DC, <<http://www.national-academies.org>> (May 1, 2006).
- Gingrich, N. (2006). "Why New Orleans Needs Saving." *Time Magazine*, New York, 03/06/2006.
- Hallowell, C. (2005). *Holding Back the Sea, The Struggle On the Gulf Coast to Save America*. Harper Collins Publishers, New York, NY.
- Houck, O. A. (2005). "Chapter 5: Environmental Protection and Sustainable Development." *Report to Mayor Nagin's Bring New Orleans Back Commission: An Alternative Vision for Rebuilding, Redevelopment & Reconstruction*, From the Lake to River Foundation, New Orleans, LA, <<http://www.fromthelaketotheriver.org>> (May 9, 2006).
- Houck, O.A. (2006). "Can We Save New Orleans?" *Tulane Environmental Law Journal*, 19(1), <<http://www.law.tulane.edu/tuexp/journals>> (Apr. 25, 2006).
- Kelman, A. (2003). *A River and Its City, The Nature of Landscape in New Orleans*, University of California Press, Berkeley, CA.
- Lopez, J.A. (2006). *The Multiple Lines of Defense Strategy to Sustain Louisiana's Coast*, Lake Pontchartrain Basin Foundation, Metairie, LA, <<http://www.saveourlake.org>> (May 9, 2006).
- Louisiana Coastal Wetlands Conservation and Restoration Task Force (1998). *Coast 2050: Toward a Sustainable Coastal Louisiana*, Louisiana Department of Natural Resources, Baton Rouge LA.
- National Research Council (2006). *Drawing Louisiana's New Map, Addressing Land Loss in Coastal Louisiana*, Committee on the Restoration and Protection of Coastal Louisiana, Ocean Studies Board, The National Academies Press, Washington DC.
- Netherlands Water Partnership (2005). *Dutch Expertise Water Management & Flood Control*, Delft, the Netherlands, <<http://www.nwp.nl>> (Feb. 3, 2006).
- Sparks, R. E. (2006). "Rethinking, Then Rebuilding New Orleans." *Issues in Science and Technology*, National Academy Press, Washington DC, Winter, 33-39.
- U.S. Army Corps of Engineers (2004). *Louisiana Coastal Area (LCA), Louisiana Ecosystem Restoration Study*, Final, Vol. 1, LCA Study - Main Report, New Orleans District.
- Working Group for Post-Hurricane Planning for the Louisiana Coast (2006). *A New Framework for Planning the Future of Coastal Louisiana after the Hurricanes of 2005*, University of Maryland Center for Environmental Science, <<http://www.umces.edu/la-restore>> (Apr. 23, 2006).
- Zinn, J. (2004). *Coastal Louisiana: Attempting to Restore and Ecosystem*, Congressional Research Service Report for Congress, Washington DC, October.
- Zinn, J. (2005a). *Coastal Louisiana Ecosystem Restoration: The Recommended Corps Plan*, Congressional Research Service Report for Congress, Washington DC, April.
- Zinn, J. (2005b). *Hurricanes Katrina and Rita and the Coastal Louisiana Ecosystem Restoration*, Congressional Research Service Report for Congress, Washington DC, September.

MULTIPLE LINES OF DEFENSE

Independent engineers and scientists are proposing multiple lines of defense as a strategy for integrating coastal restoration and hurricane protection using natural and man-made barriers

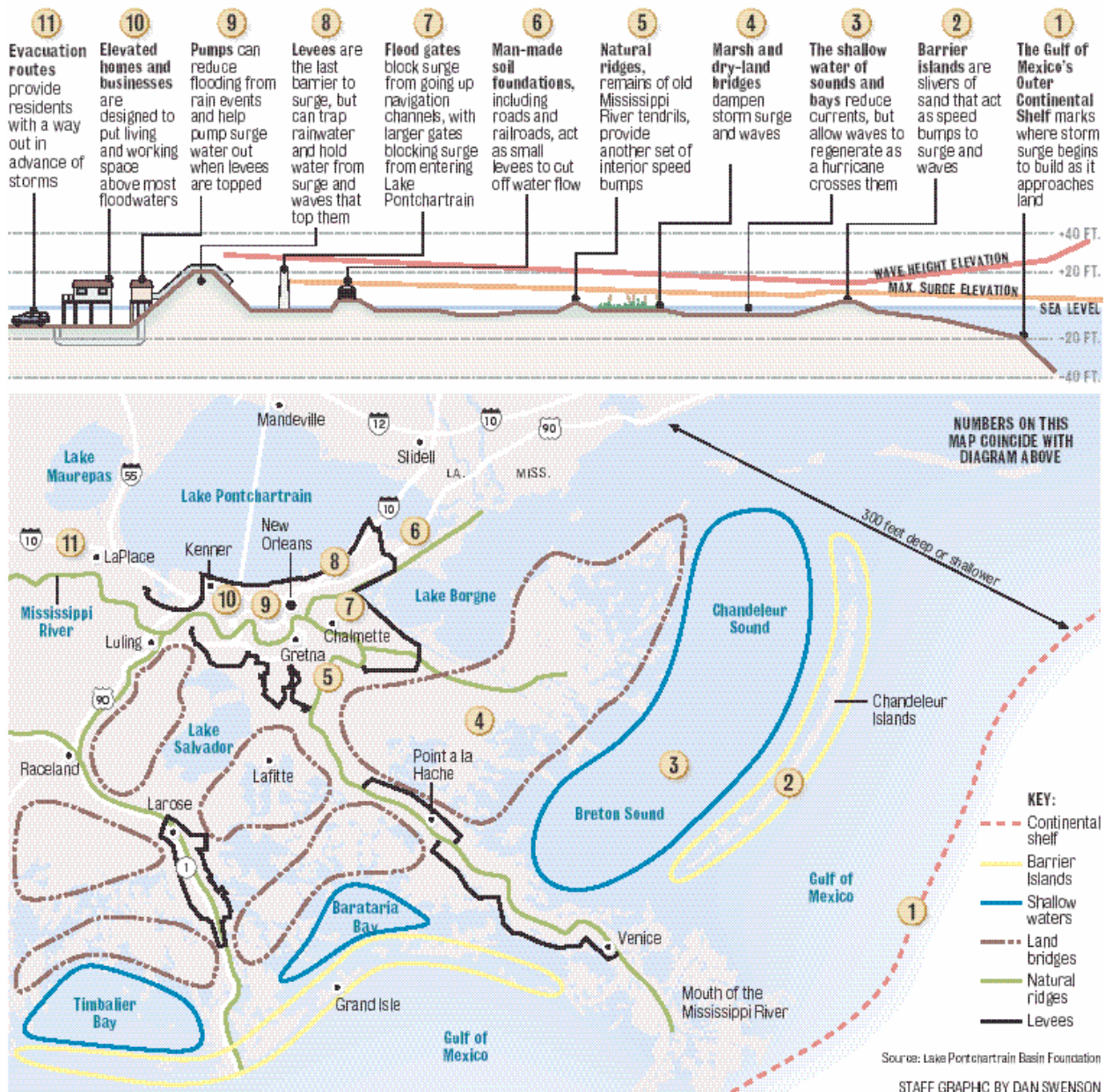


Figure 14.1: Eleven Lines of Defense (Lopez, 2006; graphic provided by the New Orleans Times Picayune)

CHAPTER FIFTEEN: FINDINGS AND RECOMMENDATIONS

15.1 Overview

This report presents the results of an investigation of the performance of the New Orleans regional flood protection system during and after Hurricane Katrina, which struck the New Orleans region on August 29, 2005. This event resulted in the single most costly catastrophic failure of an engineered system in history. Current damage estimates at the time of this writing are on the order of \$100 to \$200 billion in the greater New Orleans area, and the official death count in New Orleans and southern Louisiana at the time of this writing stands at 1,293, with an additional 306 deaths in nearby southern Mississippi. An additional approximately 300 people are currently still listed as “missing”, and the death toll is expected to continue to rise a bit further. More than 450,000 people were initially displaced by this catastrophe, and at the time of this writing more than 200,000 residents of the greater New Orleans metropolitan area continue to be displaced from their homes by the floodwater damages from this storm event.

This investigation has targeted three main questions as follow: (1) What happened?, (2) Why?, and (3) What types of changes are necessary to prevent recurrence of a disaster of this scale again in the future?

In the end, it is concluded that many things went wrong with the New Orleans flood protection system during Hurricane Katrina, and that the resulting catastrophe had its roots in three main causes: (1) a major natural disaster (the Hurricane itself), (2) the poor performance of the flood protection system, due to localized engineering failures, questionable judgments, errors, etc. involved in the detailed design, construction, operation and maintenance of the system, and (3) more global “organizational” and institutional problems associated with the governmental and local organizations responsible for the design, construction, operation, maintenance and funding of the overall flood protection system.

15.2 Performance of the Regional Flood Defense System During Hurricane Katrina

As Hurricane Katrina initially approached the coast, the resulting storm surge and waves rose over the levees protecting much of a narrow strip of land on both sides of the lower Mississippi River extending from the southern edge of New Orleans to the Gulf of Mexico. Most of this narrow protected zone, Plaquemines Parish, was massively inundated by the waters of the Gulf.

The eye of the storm next proceeded to the north, on a path that would take it just slightly to the east of New Orleans.

Hurricane Katrina has been widely reported to have overwhelmed the eastern side of the New Orleans flood protection system with storm surge and wave loading that exceeded the levels used for design of the system in that area. That is a true statement, but it is also an incomplete view. The storm surge and wave loading at the eastern flank of the New Orleans flood protection system was not vastly greater than design levels, and the carnage that resulted

owed much to the inadequacies of the system as it existed at the time of Katrina's arrival. Some overtopping of levees along the eastern flank of the system (along the northeastern frontage of the St. Bernard and Ninth Ward protected basin, and at the southeast corner of the New Orleans East protected basin), and also in central areas (along the GIWW channel and the IHNC channel) was inevitable given the design levels authorized by Congress and the surge levels produced in these areas by the actual storm. It does not follow, however, that this overtopping had to result in catastrophic failures and breaching of major portions of the levees protecting these areas, nor the ensuing catastrophic flooding of these populous areas.

The northeast flank of the St. Bernard/Ninth Ward basin's protective "ring" of levees and floodwalls was incomplete at the time of Katrina's arrival. The critical 11 mile long levee section fronting "Lake" Borgne (which is actually a Bay, connected directly to the Gulf of Mexico) was being constructed in stages, and funding appropriation for the final stage had long been requested by the U.S. Army Corps of Engineers (USACE), but this did not arrive before Katrina struck. As a result, large portions of this critical levee frontage were several feet below final design grade. In addition, an unfortunate decision had been made to use local dredge spoils from the excavation of the adjacent MRGO channel for construction of major portions of the levees along this frontage. The result was that major portions of these levees were comprised of highly erodable sand and lightweight shell sand fill.

When the storm surge arrived, massive portions of these levees eroded catastrophically and the storm surge passed through this frontage while still on the rise, crossed an open swamp area that should have safely absorbed most of the overtopping flow from the outer levees (if they had not catastrophically eroded), and it then crossed easily over a secondary levee of lesser height that had not been intended to face a storm surge largely undiminished by the minimal interference of the too rapidly eroded outer levees fronting Lake Borgne. The resulting carnage in St. Bernard Parish was devastating, as the storm surge rapidly filled the protected basin to an elevation of approximately +12 feet above sea level; deeply inundating even neighborhoods with ground elevations well above sea level in this area.

The storm-surge-swelled waters of Lake Borgne also passed over and then through a length of levees at the southeast corner of the New Orleans East protected basin. Here too, the levees fronting Lake Borgne had been constructed in part using materials dredged from the excavation of an adjacent shipping channel (the GIWW channel), and these levees also contained significant volumes of highly erodable sands and lightweight shell sands. These levees also massively eroded, and produced the principal source of flooding that eventually inundated the New Orleans East protected area. Here again, there was an area of undeveloped swampland behind the outer levees that might have helped to absorb the brunt of any overtopping flow, and a secondary levee of lesser height was in place behind this swampland that might then have prevented or at least greatly slowed and reduced the catastrophic flooding of the populous areas of New Orleans East. This secondary levee was not able to resist the massive flows resulting from the catastrophic erosion of the highly erodable sections of the Lake Borgne frontage levee, however, and some of the eroded and breached frontage levees allowed waters to bypass the secondary levee line. As a result, the floodwaters from the breaches and eroded sections of levee at the southeast corner of the New Orleans East protected area passed inland and began the filling of the New Orleans East protected basin.

The catastrophic erosion of these two critical levee frontages need not have occurred. These frontages could instead have been constructed using well compacted clay fill with good resistance to erosion, and they could have been further armored in anticipation of the storm surge and wave loading from Lake Borgne. The levee at the northeast edge of St. Bernard Parish could have been completed in a more timely manner. The result would have been some overtopping, but not catastrophic erosion and uncontrolled breaching of these critical frontages. Some flooding and damage would have been expected, but it need not have been catastrophic.

The storm surge swollen waters of Lake Borgne next passed laterally along the east-west trending GIWW/MRGO channel to its intersection at a “T” with the north-south oriented IHNC channel, overtopping levees along both banks to a limited degree. This produced an additional breach of a composite earthen levee and concrete floodwall section (at a transition to a full earthen levee section) along the southern edge of New Orleans East, adding additional uncontrolled inflow to this protected basin. This failure could have been prevented at little incremental cost if erosion protection (e.g. a concrete splash pad, or similar) had been emplaced along the back side of the concrete floodwall at the levee crest, but the USACE felt that this was precluded by Federal rules and regulations regarding authorized levels of protection.

The surge next raised the water levels within the IHNC channel, and produced a number of failures on both the east and west banks. Two major failures occurred on the east side of the IHNC, at the west edge of the Ninth Ward. Overtopping occurred at both of these locations, but this was not the principal cause of either of these failures. Both failures were principally due to underseepage flows that passed beneath the sheetpile curtains supporting the concrete floodwalls at the crests of the levees. Like many sections of the flood protection system, these sheetpiles were too shallow to adequately cut off, and thus reduce, these underseepage flows. The result was two massive breaches that devastated the adjacent Ninth Ward neighborhood, and then pushed east to meet with the floodwaters already rapidly approaching from the east from St. Bernard Parish as a result of the earlier catastrophic erosion of the Lake Borgne frontage levees.

Several additional breaches also occurred farther north on the east side of the IHNC fronting the west side of New Orleans East, but these were relatively small features and they just added further to the uncontrolled flows that were now progressively filling this protected basin. These breaches occurred mainly at junctures between adjoining, dissimilar levee and floodwall sections, and represented good examples of widespread failure to adequately engineer these “transitions” between sections of the regional flood protection system.

Several breaches occurred on the west side of the IHNC, and these represented the first failures to admit uncontrolled floodwaters into the main metropolitan (downtown) protected area of New Orleans. These features did not scour and erode a path below sea level, however, so they admitted floodwaters for a number of hours and then these inflows ceased as the storm surge in the IHNC eventually subsided. Only 10% to 20% of the floodwaters that eventually inundated a majority of the main (downtown) New Orleans protected basin entered through these features.

These failures and breaches on the west side of the IHNC all appear to have been preventable. One failure was the result of overtopping of an I-wall, with the overtopping flow then eroding a trench in the earthen levee crest at the inboard side of the floodwall. This removal of lateral support unbraced the floodwall, and it was pushed over laterally by the water pressures from the storm surge on the outboard side. Here again the installation of erosional protection (e.g. concrete splash pads or similar) might have prevented the failure.

The other failures in this area occurred at “transitions” between disparate levee and floodwall sections, and/or at sections where unsuitable and highly erodable lightweight shell sand fills had been used to construct levee embankments. Here, again, these failures were as much the result of design choices and/or engineering and oversight issues as the storm surge itself.

Particularly frustrating were a pair of failures on the east and west banks of the IHNC where the CSX railroad crossed the IHNC. These two sites both breached as a result of improper detailing of the intersections between the railroad tracks and their support gravel ballast, and adjacent roadways also crossing the federal levees at these same two locations. These represented additional examples of repeated problems associated with coordination, design, and oversight of complex “intersections” wherein multiple agencies and utilities (including roadways, rail lines, etc.) intersect the protective levee system. Frustratingly, it is noted that these same two rail crossings at the east and west sides of the IHNC had also failed and breached in 1965, during hurricane Betsy.

As the eye of the hurricane next passed to the northeast of New Orleans, the counterclockwise swirl of the storm winds produced a storm surge against the southern edge of Lake Pontchartrain. This produced additional temporary overtopping of a long section of levee and floodwall at the west end of the lakefront levees of New Orleans east, behind the old airport, adding further to the flows that were progressively filling this protected basin.

The surge against the southern edge of Lake Pontchartrain also elevated the water levels within three drainage canals at the northern edge of the main metropolitan (downtown) New Orleans protected basin, and this would produce the final, and most damaging, failures and flooding of the overall event.

The three drainage canals should not have been accessible to the storm surge. The USACE had tried for many years to obtain authorization to install floodgates at the north ends of the three drainage canals that could be closed to prevent storm surges from raising the water levels within the canals. That would have been the superior technical solution. Dysfunctional interaction between the local Levee Board (who were responsible for levees and floodwalls, etc.) and the local Water and Sewerage Board (who were responsible for pumping water from the city via the drainage canals) prevented the installation of these gates, however, and as a result many miles of the sides of these three canals had instead to be lined with levees and floodwalls.

The lining of these canals with levees topped with concrete floodwalls was rendered very challenging due to (a) the difficult local geology of the foundation soils, and (b) the narrow right of way (or available “footprint”) for these levees. As a result of the decision not

to install the floodgates, the three canals represented potentially vulnerable “daggers” pointed at the heart of the main metropolitan New Orleans protected basin. Three major breaches would occur on these canals; two on the London Avenue Canal and one on the 17th Street Canal. All three of these breaches eroded and scoured rapidly to well below sea level, and these three major breaches were the source of approximately 80 to 85% of the floodwaters that then flowed into the main (downtown) protected basin over the next three days, finally equilibrating with the still slightly elevated waters of Lake Pontchartrain on Thursday, September 1.

The central canal of the three, the Orleans Canal, did not suffer breaching, but a section of floodwall topping the earthen levee approximately 200 feet in length near the south end of the canal had been left incomplete, again as a result of dysfunctional interaction between the local levee board and the water and sewerage board. This effectively reduced the level of protection for this canal from about +12 to +13 feet above sea level (the height of the tops of the floodwalls lining the many miles of the canal) to an elevation of about +7 feet above sea level (the height of the earthen levee crest along the 200 foot length where the floodwall that should have topped this levee was omitted). As a result of the missing floodwall section, flow passed through this “hole” and began flowing into the heart of the main New Orleans protected basin. This flow eventually ceased as the storm surge subsided, and so was locally damaging but not catastrophic.

The three breaches on the 17th Street and London Avenue canals were catastrophic. None of these failures were the result of overtopping; surge levels in all three drainage canals were well below the design levels, and well below the tops of the floodwalls. Two of these breaches were the result of stability failures of the foundation soils underlying the earthen levees and their floodwalls, and the third was the result of underseepage passing beneath the sheetpile curtain and resultant catastrophic erosion near the inboard toe of the levee that eventually undermined the levee and floodwall.

A large number of engineering errors and poor judgments contributed to these three catastrophic design failures, as detailed in Chapter 8. In addition, a number of these same problems appear to be somewhat pervasive throughout other areas of the New Orleans regional flood defense system(s), and call into question the integrity and reliability of other sections of the regional flood protection system that did not fail during this event. Indeed, additional levee and floodwall sections along the drainage canals appear to have been potentially heading towards failure when they were “saved” by the occurrence of the three large breaches (which rapidly drew down the canal water levels and thus reduced the loading on nearby levee and floodwall sections.)

15.3 Engineering Issues

The New Orleans regional flood protection system failed at many locations during Hurricane Katrina, and by many different modes and mechanisms. This unacceptable performance can in many cases be traced to engineering lapses, poor judgments, and efforts to reduce costs at the expense of system reliability. These, in turn, were to a large degree the result of more global underlying “organizational” and institutional problems associated with the governmental and local organizations jointly responsible for the design, construction,

operation, and maintenance of the flood protection system, including provision of timely funding and other critical resources.

Our findings to date indicate that no one group or organization had a monopoly on responsibility for the catastrophic failure of this regional flood protection system. Many groups, organizations and even individuals had a hand in the numerous failures and shortcomings that proved so catastrophic on August 29th. It is a complex situation, without simple answers.

It is not without answers and potential solutions, however, just not simple ones. There is a need to change the process by which these types of large and critical protective systems are created and maintained. It will not be feasible to provide an assured level of protection for this large metropolitan region without first making significant changes in the organizational structure and interactions of the national and more local governmental bodies and agencies jointly responsible for this effort. Significant changes are also needed in the engineering approaches and procedures used for many aspects of this work, for the standards used in such design, in the conceptual approaches considered, and in the conceptualization and engineering treatment of potential modes of failure and poor performance during design, construction and operation. There is also a need for interactive and independent expert technical oversight and review as well. In numerous cases, it appears that such review would have likely caught and challenged errors and poor judgments (both in engineering and in policy and funding) that led to failures during Hurricane Katrina.

There are many detailed engineering lessons developed within this report, but a number of overarching engineering issues have been identified, and a number of the most important of these are presented below. These are a somewhat urgent set of issues, as the USACE and the IPET investigation are currently working to assess the level of risk associated with the now largely reconstructed system, and these issues impact that assessment.

1. Overall levels of safety and reliability targeted during engineering design and analysis were inappropriately low for a critical system protecting a major metropolitan area. Factors of safety and analysis methods and procedures used in design calculations for the “transient” loading conditions associated with hurricane-induced storm surge, coupled with the design surge elevations employed, provided levels of risk that were on the order two to three orders of magnitude higher than the standards generally used in U.S. dam practice where similarly large populations are at potential risk. This left too little room for error, uncertainties, or surprises.
2. The difficult and complex geology of the region posed design challenges that were not adequately addressed. Insufficient site investigation and characterization of foundation soil conditions at many sites produced minor short-term project savings, but these pale against the massive losses that ensued. More attention needs to be paid to the geology, and more detailed site investigation and site characterization is clearly warranted given the potential consequences of failures.

3. There was a persistent pattern of attempts to reduce costs of constructed works, at the price of corollary reduction in safety and reliability. This represented a policy that has now been shown to be massively “penny wise and pound foolish”.
4. A pattern of optimistic engineering assessment with regard to a number of potential sources of risk and of potential modes of failure was endemic to the detailed design of a number of major system elements. This included:
 - (a) The risks associated with underseepage flows during “transient” storm surges were systematically underestimated. This led to the use of sheetpile curtains that were extended to inadequate depths at a number of locations, and it led directly to a number of the major failures and breaches during hurricane Katrina. Appropriate consideration and analysis of underseepage issues (including potential embankment instability due to pore pressure induced strength reduction, and potential erosion and piping) for transient storm surge conditions was routinely missing, and the overall system should now be re-evaluated with regard to these underseepage-related potential modes of failure.
 - (b) The use of highly erodable sand and even lightweight shell sand fills in levee sections also figured prominently at numerous locations of breaching and catastrophic erosion. Use of such materials should henceforth be disallowed in this system that protects a major metropolitan region. Here again, the overall system should be re-evaluated for their presence, and the levels of risk posed by the presence of these unsuitable materials, both in levee embankments and at shallow depths within the underlying foundation soils; and this risk should be mitigated.
 - (c) Similarly, design procedures did not include consideration of the potential failure mode that involves formation of a ‘gap’ at the outboard side of the floodwalls, between the outboard section of the earthen levee embankment and the sheetpile curtains supporting the floodwalls. Formation of such gaps occurred at a number of sites as pressure increased on the outboard sides of the floodwalls, and water then intruded into the gaps and greatly increased the lateral “push” of the storm surge (water) against the sheetpile/floodwalls. A number of failures occurred as a result. In the future, such gapping should be “assumed” during analysis and design. Many of the “I-wall” type concrete floodwalls are currently being removed and replaced by the more robust “T-wall” type floodwalls (which have additional battered piles to help them resist overturning and lateral displacement.) These T-wall systems will have somewhat increased capacity, but they too will need to be analyzed with regard to this potential failure mode. It cannot simply be assumed that “T-walls” are intrinsically completely safe.
5. Design review was generally inadequate, and there was an institutional failure to catch and challenge unconservative design assumptions and interpretations of data, misconceptions, poor judgements, and errors. Instigation of interactive consultation and review by consulting panels of leading outside experts is common practice in dam engineering. It should be common practice in levee engineering as well; especially when the levee systems protect significant populations. In addition, it would be wise

for local interests (e.g. the State and/or the City) to mount an additional unbiased expert review panel (again including leading outside experts) to provide a second check and opinion. At many failure sites it appears likely that suitable expert review would have caught and challenged errors and questionable judgments that contributed to the failures observed.

6. Improved advantage needs to be taken of ongoing technical advances related to the engineering, design and construction of these types of regional flood defense systems. Engineering design concepts and analysis approaches employed at many locations were sorely “outdated” at the time of their use, and there was a lack of movement towards embracing new and improved methods and tools. “This is how we have always done it” is a potentially dangerous concept here, and inertia in terms of embracing technical advances was a troubling issue. Failure to embrace their own full-scale field testing and research led the Corps to neglect the “water-filled gap” as a potential failure mode to be addressed during design. And it is time to relegate the “Method of Planes” to its place in history and to adopt more modern and more flexible stability analysis methods capable of addressing a wider range of geometries and potential failure modes.
7. The USACE is the lead oversight agency with regard to engineering and construction of the regional flood defense system. The Corps needs to be allocated adequate funding and support, given the ability to perform research, and granted adequate freedom and support to facilitate the continuing professional development (and retention) of highly qualified engineers within the Corps in order to ensure an adequate in-house supply of engineering expertise for their critical role.

15.4 Looking Back - Organized for Failure

The ILIT mandate at the outset of this investigation study was to include study of historical and organizational - institutional issues, political and budgetary considerations, decision making, utilization of technology, and the evolving societal, governmental, and organizational priorities over the life of the Flood Defense System for the Greater New Orleans Area (NOFDS). One cannot understand the failure of the NOFDS without understanding both the underlying engineering and organizational mechanics that were interwoven in the evolution of this failure.

ILIT's view of the importance of these *organizational, institutional, resource and technology delivery* factors increased during the course of this study. These factors are grouped into what is termed a *Technology Delivery System* (TDS). A TDS can be represented as system that has organizational components, inputs, outputs, and information linkages that are interactive, inter-dependent, and adaptive. Three primary organizational components comprise a TDS for a system such as the NOFDS. These are: (1) society (the public), (2) government (federal, state, local), and (3) enterprise (commercial, industrial, private). These components are embedded in and interact with their natural and cultural environments. Inputs comprise knowledge plus human, natural, and fiscal resources. Outputs include desired goods or services and undesired outcomes or unintended consequences.

Eight principal categories of TDS malfunctions were identified that played major roles in the catastrophic failure of the NOFDS, and these are as follow:

Failures of foresight: Catastrophic flooding of the greater New Orleans area due to surge from an intense hurricane was predicted for several decades. The consequences observed in the wake of hurricane Katrina were also predicted. The hazards associated with the NOFDS were not adequately recognized, defensive measures were not identified and prioritized, and effective action was not mobilized to effectively deal with these hazards.

Failures of organization: The roots of the failure of the NOFDS are firmly embedded in flawed organizational - institutional systems. The organizational - institutional systems lacked centralized and focused responsibility and authority for providing adequate flood protection. There were dramatic and pervasive failures in management represented in ineffective and inefficient planning, organizing, leading, and controlling to achieve desirable quality and reliability in the NOFDS. There were extensive and persistent failures to demonstrate initiative, imagination, leadership, cooperation, and management.

Failures of resource allocation: Contributing to the failure of the NOFDS was provision of inadequate resources based primarily on recommendations provided by the Corps of Engineers. This was followed by failure of the federal and state governments to fund badly needed improvements once limitations were recognized. In a number of instances, State and/or local agencies pressured for 'lower cost' solutions not realizing that these solutions would result in lowering the overall quality and reliability of the NOFDS. There were important deficiencies in the cost - benefit analyses used to justify the levels of protection (and reliability) provided, and also the continued improvement in these levels of protection (and reliability) as knowledge and technology advanced.

Failures of diligence: Forty years after the devastating flooding caused by hurricane Betsy, the flood protection system authorized in 1965 and based on the Standard Project Hurricane (SPH) was still not completed when hurricane Katrina arrived. In addition, the concept and application of the SPH was recognized to be seriously flawed, yet there were no adjustments made to the system to address this before Katrina struck. Early warning signs of deficiencies and flaws persisted throughout progressive development and construction of the different components that comprised the NOFDS, and these warning signs were not adequately evaluated and acted upon.

Failures of decision making: The history of this system was marked by a series of flawed decisions and trade-offs that proved to be fatal to the ability of the system to perform adequately. Compromises in the ability of this system to perform adequately started with the decisions regarding the fundamental design criteria for the development of the system, then were propagated through time as alternatives for the system were evaluated and engineered. Design, construction, operation, and maintenance of the system in a piecemeal fashion allowed the introduction of additional flaws and defects. Efficiency was traded for effectiveness. Superiority in provision of an adequate NOFDS was traded for mediocrity, lower expenditures, and getting along.

Failures of management: Requirements imposed on the Corps of Engineers by Congress, the White House, State and local agencies, and the general public have changed dramatically during the past three decades. Defense, re-construction, maintenance, waste disposal, recreational development, emergency response, and ecological restoration have served to divert attention from flood control. Public and Congressional pressures to (1) reduce backlogs of approved projects, (2) improve project and organizational efficiency (e.g.: down-sizing, out-sourcing, etc.), (3) address environmental impacts, and (4) develop appropriations for projects have served to divert attention from engineering quality and reliability of flood control. Engineering technology leadership, competency, expertise, research, and development capabilities appear to have been sacrificed for improvements in project planning and controlling.

Failures of synthesis: While individual parts of a complex system can be adequate, when these parts are joined together to form an interactive - interdependent - adaptive system, unforeseen failure modes can be expected to develop. These unforeseen, but foreseeable, failure modes did develop in the NOFDS during hurricane Katrina. It is evident that insufficient attention was given to creation of an integrated series of components to provide a reliable overall NOFDS. Synthesis was subverted to decomposition, as projects were engineered and constructed in piecemeal fashion to conform to incremental appropriations. As a result, many failures developed at interfaces or 'transitions' in the NOFDS.

Failures of risk assessment and risk management: The risks (likelihoods and consequences) associated with hurricane surge and wave induced flooding were seriously underestimated. There was inadequate recognition of the primary contributors to the likelihoods and consequences of catastrophic flooding. Sufficient defensive measures to counteract and mitigate these uncertainties were not employed. Factors of safety used in design of the primary elements in the NOFDS were not sufficient; and represented implicit levels of system reliability that were inappropriately low for a system protecting a major metropolitan region. Quality assurance and quality control measures invoked during the life of the system failed to disclose critical flaws in the system. Inappropriate use was made of existing engineering technology available to design, construct, operate, and maintain a NOFDS that would have acceptable quality and reliability. Deficient risk management methods were used to allocate resources and impel action to properly manage risks. Risk management failed to employ continuing improvement, monitoring, assessment, and modifications in means and methods which were discovered to be ineffective.

15.5 Looking Forward - Organizing for Success

The following recommendations are offered for consideration in developing a NOFDS that will have desirable and acceptable quality and reliability. These recommendations are divided into two categories: engineering developments and organizational developments. It will take both, working together, to realize the desired goals of an appropriately improved NOFDS. The primary challenge is timely mobilization of inspired and inspiring leadership, adequate resources, existing technology, and high reliability organizations.

15.5.1 Strategic and Engineering System Issues:

The technology exists that can be used to develop a NOFDS that will be effective and efficient. A major challenge is timely and proper application of this technology.

Recommendation 1: Develop an integrated and coherent Flood Defense System for the greater New Orleans area (NOFDS) that will provide desirable and acceptable levels of flood protection throughout its life-cycle. Particular attention must be paid to interfaces and interdependencies in this system. The NOFDS should be balanced, complete, cohesive, clear, consistent, and have controls and continuity. The NOFDS should be based on the best available and safest technology and most up-to-date legal standards. Risks should be properly identified, contained and compartmentalized. The system must recognize the unique natural environmental setting including its geology, meteorology, oceanography, the Mississippi River floodplains, deltas and wetlands, subsidence, and the rise in sea level and frequency and intensity of hurricanes. The system must also recognize and accommodate the unique societal and cultural environments of this area.

Recommendation 2: Develop a NOFDS based on enhancing natural defenses supplemented with engineered defenses that incorporate concepts of defenses in depth, robustness or resilience, and fail-safe performance. Selective re-establishment of natural coastal defenses and wetlands, and restored floodplains to provide for river floods, should be supplemented with engineering works that together will have the capabilities of providing desirable and acceptable levels of flood protection. Coastal management must be focused on providing safety from flooding and environmental protection. Water should be given space. Some areas will have to be returned to nature, and judicious and wise decisions will have to be reached regarding which areas will be populated and developed and the levels of protection that will be provided to these areas. Engineering works should include: (1) raising, strengthening, improving the reliability, and improvement of the erosion resistance of levees, (2) provision of floodgates, and storm surge barriers, (3) improved positioning and defense of modern pump stations, (4) compartmentation to limit potential flooding consequences, and (5) adequate and effective evacuation measures to help limit effects on people and their possessions. A robust NOFDS will require a combination of appropriate configuration of engineered elements and components, ductility or an ability to deform and stretch and not lose important performance characteristics (e.g. the ability to overtop for some limited period of time without catastrophic breaching), and provision of excess capacity so that if some elements or components are overloaded or do not perform desirably then desirable protection can still be maintained. Fail safe characteristics should be provided in all of the important elements of the NOFDS so that when the design and ultimate performance conditions are exceeded, the performance characteristics are not excessively compromised.

Recommendation 3: Develop a NOFDS founded on advanced Risk Assessment and Risk Management principles for all phases in the life-cycle including concept development, design, construction, operation, and maintenance. These principles should address natural processes, analytical modeling, human and organizational performance, and knowledge acquisition and utilization uncertainties and be based on proactive, reactive, and interactive risk assessment and management approaches. These approaches should be based on reductions in likelihoods of failure, reduction in the consequences associated with potential

failures, and improvements in detection and correction of developments that can lead to failures. Advanced Risk Assessment and Risk Management approaches should be used to provide decision makers with information to define what levels of protection should be provided for which areas, and how much can and should be spent for those purposes.

Recommendation 4: Develop updated engineering guidelines and procedures for all elements and components to be incorporated in the FDS for all life-cycle phases based on proven state-of-practice and state-of-art technology. Where technology gaps are identified, then substantial development programs should be implemented to fill these gaps with existing research results. Where technology gaps cannot be filled with existing research results, then research should be undertaken or sponsored to enable timely filling of the technology gaps. Upgrading the technical capabilities of the engineers responsible for oversight and design, and the use of interactive boards of consultants as well as expert external review boards, would likely greatly improve the ability to deliver reliable flood protection.

Recommendation 5: Develop, implement, and enforce advanced Quality Assurance and Quality Control methods and procedures for all life-cycle phases of the NOFDS. Quality Assurance (proactive) and Quality Control (interactive) measures are of particular importance to help disclose 'predictable surprises' and variances in the desirable quality characteristics of the elements and components in the NOFDS. These methods and procedures should be used in all life-cycle phases of the NOFDS including concept development, design, construction, operation, maintenance, and continued improvement. These procedures and measures need to assure that the best available and safest technology is being used and used properly.

15.5.2 Technology Delivery System Developments - Organizing for Success

It will not be feasible to create an adequately reliable regional Flood Defense System without addressing the organizational, institutional, political and resources issues that adversely affect the current process. Simply changing engineering procedures, design manuals, and the review process will not suffice.

The primary requirement for reconstitution of a Technology Delivery System that can and will provide an adequate and acceptably reliable NOFDS is mobilization of the 'will' to provide such a system. If the United States decides that the catastrophe of Katrina will not be repeated, then the necessary leadership, organization, management, resources, and public support must be mobilized to assure such an outcome. One of the primary challenges is time; the clock is ticking until this area of the United States is again confronted with a severe challenge of flooding.

Recommendation 1: Seriously consider defining risk in the framework of federal, state, and local government responsibilities to protect their citizens.

Recommendation 2: Exploit the major and unprecedented role that exists for citizens who should be considered part of governance in the spirit that those who govern do so at the informed consent of the governed. This is the population exposed to catastrophic risks and the people that will be protected by the NOFDS. Authorities responsible for catastrophic risk management should ensure that those vulnerable have sufficient and timely information regarding their condition, and a reciprocal ability to respond to requests for their informed

consent especially regarding tradeoffs of safety for cost. The public protected by the NOFDS need to be encouraged to actively and intelligently interact with its development.

Recommendation 3: Intensify, focus, and fund Corps of Engineers reorganization and modernization efforts directed toward (1) increasing and maintaining in-house engineering capabilities and project performance, (2) increasing in-house research and development capabilities, (3) increasing in-house engineering performance on technically challenging projects, (4) developing an organizational culture of high reliability founded on existing organizational cultural values of Duty, Honor, Country, and (5) developing a leadership role and responsibility for technical and management oversight of all phases in development of a NOFDS. Technical superiority must be re-established. Outsourcing must be balanced with in-sourcing to encourage development and maintenance of superior technical leadership and capabilities within the USACE. This will require close and continuous collaboration of federal legislative, executive, and judicial agencies. This will require that the USACE reconceptualize itself as a pivotal part of a modular organization developing partnerships with other federal agencies, state and local governments, enterprise interests, and private stake holders. This will require additional funding; in the end the nation will get only what it is willing to invest and pay for.

Recommendation 4: Restructure federal/state relationships in flood control. One possible model is what has been called “modularity” -- a concept which involves provisional and functional rearrangement of units in terms of alternative configurations of tools, structures and relationships.

Recommendation 5: Develop a National Flood Defense Authority (NFDA) charged with oversight over the design, construction, operation and maintenance of flood control systems. Each state would have an equivalent organization that could foster cooperation and developments between and within the states. The Corps of Engineers, state flood control authorities, and technical advisory boards would work with the NFDA to foster application of the best available technology and help coordinate development and maintenance efforts and planning. In cooperative developments, federal and state governments would provide reliable and sustainable funding for the life-cycle of specific flood defense systems. This development should be accompanied by development of an integrated and coherent Louisiana Flood Defense Authority representing state, regional, local, city, and public stakeholders that can focus and prioritize stakeholder interests and requirements and collaborate with the Corps of Engineers in development of a NOFDS.

Recommendation 6: Because of the importance of emergency response in the NOFDS, FEMA should be developed as a high reliability organization and returned by the executive branch to Cabinet level status. A new Council for Catastrophic Risk Management should be appointed in the White House and given oversight of disaster preparation and response. A similar body should be appointed to Congress. Incentives must be created to encourage all levels of government to responsibly deal with potential national, regional, and local catastrophes.

15.6 Conclusion

The performance of the New Orleans regional flood protection system during hurricane Katrina was unacceptable. Detailed study has now led to understanding of the physical causes and mechanisms of most of the many failures and breaches, and this in turn provides a basis for development of improved conceptual and engineering design methods, as well as improved review and overview paradigms.

Simply addressing engineering design methods, standards and procedures is unlikely to be sufficient to provide a suitably reliable level of protection, however. There is also a need to resolve difficult issues intrinsic in the operations and relationships between (1) Federal and more local government as they serve as decision-making, policy and funding sources, (2) the Federal and local agencies responsible for the actual design, construction, operation and maintenance of such flood protection systems, and (3) private enterprise that must assist in construction. Some of these groups need to enhance their technical capabilities; a long-term expense that would clearly represent a prudent investment at both the national and local level, given the stakes as demonstrated by the massive losses in this recent event. Steady commitment and reliable and sustainable funding, shorter design and construction timeframes, clear lines of authority and responsibility, and improved overall coordination of disparate system elements and functions are all needed as well.

The overall philosophy and basis for design of these types of expensive and vital systems warrants reconsideration. Improvements such as (1) conceptual design strategies that involve working in conjunction with natural barriers and other favorable features, (2) system-based risk assessment, analysis and design, (3) allocation of appropriate resources, (4) embracing research and appropriate technological advances, and (5) maintenance of a deliberate culture of diligence in seeking overall system reliability would all represent significant steps forward.

And there is some urgency to all of this. The greater New Orleans regional flood protection system was significantly upgraded in response to flooding produced by Hurricane Betsy in 1965. The improved flood protection system was intended to be completed in 2017, fully 52 years after Betsy's calamitous passage. The system was incomplete when Katrina arrived. As a nation, we must manage to dedicate the resources necessary to complete projects with such clear and obvious ramifications for public safety in a more timely manner.

New Orleans has now been flooded by hurricanes six times over the past century; in 1915, 1940, 1947, 1965, 1969 and 2005. It should not be allowed to happen again.